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SEISMIC RELIABILITY STUDY OF THE SEATTLE WATER DEPARTMENT'S WATER SUPPLY SYSTEM

Prepared for:

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1.0 EXECUTIVE SUMMARY

The Seattle Water Department (SWD) service area and watersheds lie in seismic risk zone 3, in which there is a potential for major earthquakes (7.5 Richter magnitude or larger). Due to the importance of maintaining an operable water system after a major earthquake, the likelihood of having a damaging earthquake, and the age of most of the SWD facilities, the SWD retained a seismic/structural consultant (Cygna Consulting Engineers) to assess the reliability of their water system. This assessment included a determination of seismic hazards in the Puget Sound Region, coupled with performance of seismic vulnerability assessments and development of upgrade recommendations for key facilities, including:

a) Tolt Regulating Basin, Landsburg Diversion and Lake Youngs facilities.

b) Nine water storage standpipes and seven elevated steel tanks.

c) Thirty-eight pump stations and post-chlorination treatment facilities.

d) Major pipelines, including about 150 miles of supply lines and 75 miles of major mains.

The study also included a review of the existing SWD emergency response plan for appropriateness and completeness.

One of the more important items in conducting the seismic vulnerability assessment was to establish an appropriate seismic criteria to be used in the evaluations. Based upon a combination of past precedence and historical seismicity in the Puget Sound area, it was decided to evaluate the facilities for two levels of earthquakes.

The Class I seismic event was defined as a major earthquake which had a chance of occurring about once every 500 years, and about a 10 percent chance of being exceeded in a given 50-year period. The Class II event was defined as the largest earthquake which could occur about once in every 100 years. In terms of Richter magnitude, the Class I earthquake level developed for this study was equivalent to a magnitude 7.5 event and the Class II earthquake was equivalent to a 6.5.

The Class I and Class II earthquake levels were defined in terms of percent of acceleration of gravity for three different types of soil conditions: Zone I (bedrock), Zone II (glacial sediments), and Zone III (alluvium). In considering the effect that either earthquake level had on a given facility, it was assumed that the earthquake occurred immediately under the facility in question. The study also considered a large magnitude 8.25 subduction event that has been postulated to occur off the coast. However, it was determined that the severity of the seismic motions resulting from this subduction event would reduce as the seismic waves travelled inland, and that by defining the Class I earthquake event to be an earthquake which could occur directly under the site of a given facility results in a more severe seismic criteria definition for the purposes of this study.

Because the SWD system was initiated prior to the turn of the century, many of the facilities are more than 40 to 50 years old and were designed prior to the existence of seismic design requirements in the building codes. It was not unexpected, therefore, that selected facilities would be found to be vulnerable and would require seismic upgrade measures in order to remain operable following a major earthquake.

In Table 2-1 is presented a summary of the major facilities evaluated, along with a cost estimate for the engineering and construction upgrades of these facilities.

Table 2-1

Summary of Recommended Upgrades
(Based on 1989 Dollars)

Facility	Engineering	Construction	Total
Tolt	$ 55,100	$ 87,900	$ 135,000
Landsburg	66,300	156,200	210,500
Cedar Falls	300	2,900	3,200
Lake Youngs	18,400	17,000	35,400
Control Works	62,800	121,500	176,300
Water Oper. Control Ctr.	135,000	675,800	779,800
Treatment/Pump Stations	233,500	477,300	681,800
Misc. Facilities	4,000	8,000	12,000
Pipelines	189,000	291,500	454,500
Elevated Tanks	499,000	2,871,500	3,253,500
Standpipes	532,000	3,809,000	4,221,000
Totals:	$1,795,400	$8,518,600	$10,314,000

The engineering cost estimate includes a small amount of funds (about 12%) which are set aside for additional, detailed investigations of selected Tolt, Landsburg, Lake Youngs, Control Works, Water Treatment/Pump Station and Pipeline facilities. The remaining portion is for the architectural and engineering design of the seismic upgrades. The construction cost estimate includes funding for both construction engineering and construction.

As indicative from the relative magnitudes of the remedial cost estimates for the various SWD facilities, the most highly vulnerable facilities were found to be the elevated water tanks and standpipes. The seven elevated water tanks have a total water storage capacity of 8.5 million gallons. Of these, three tanks having a capacity of 2.5 million gallons were anticipated not to remain operable following a Class II earthquake, and an additional three of the remaining elevated storage tanks would not be operable after a Class I earthquake. The nine standpipes have a total storage capacity of 8.87 million gallons. Six of the nine standpipes, having a total capacity of 5.1 million gallons, were anticipated not to remain operable following a Class II earthquake. A seventh standpipe would not be operable after a Class I earthquake.

Typically, the elevated tanks and standpipes were found to have insufficient anchorages or bracing which will require strengthening. This finding is not surprising, since all of the elevated tanks were designed and constructed between 1919 and 1959, and the standpipes between 1901 and 1933. The cost estimate for the standpipes is higher than that for the elevated tanks because the elevated tank design provides for a much more "open" type of structure that can be readily accessed to install additional bracing and anchorage. Additionally, one of the standpipes, Volunteer Park, is a historical landmark structure with an external brick enclosure which provides limited access to the tank. Until such time that the tanks and standpipes can be upgraded, the Seattle Water Department will be operating the tanks at the lowest water level that is practicable in order to minimize the seismic forces that could be generated.

The Control Works Facility was analyzed in detail for both the Class I and Class II earthquakes. For the Class I earthquake, the structure was determined to be stressed close the allowable limits. At the conclusion of this preliminary study, it was determined that the actual water elevation in the Control Works surge tanks was 5 ft. higher than what was reported on the original design drawings. Further study and analysis will be required to determine the impact of the increased water level on the integrity of the structure. All of the equipment in the Control Works Facility, which consists primarily of miscellaneous large-diameter piping and valves, was determined to be well designed and supported and capable of withstanding a Class I seismic event.

The Water Operations Control Center complex consists of the main Administration Building, along with a Warehouse, Pipe/Carpentry Shop, Vehicle Maintenance Facility and Storage Shed. The Administration Building structure was determined to be adequately designed to withstand the Class I earthquake, however it was recommended that the main control room consoles be properly anchored, along with the boiler, storage shelves, and precast plant containers on the second floor ledges. The remaining buildings, with the exception of the Storage Shed, were all found to have key lateral load-resisting structural elements which are in need of strengthening. It was recommended that the Storage Shed, which consists of unreinforced masonry and hollow clay tile construction, be replaced with a new structure.

For the remainder of the SWD facilities which were investigated, several recommendations have been made in this report which would enhance a given facility's ability to remain operable following a major earthquake. Examples of recommended upgrades include strengthening of roof diaphragms with plywood sheathing, infilling of openings in selected masonry walls to increase the lateral seismic load resisting capacity of the structure, and reinforcement of the connections of existing walls to the roof and foundation.

Recommendations were also made to properly anchor and brace selected electrical and mechanical equipment items (e.g. boilers, transformers, control consoles, motor control centers) and contents (e.g. tanks, storage racks, lockers and shelves). Many of these remedial upgrades can be implemented by SWD maintenance and construction personnel. The more major structural upgrades, such as those required for the

elevated tanks and standpipes and several of the facilities, will have to be implemented by an outside contractor.

The existing SWD emergency response plan was found to include most of the basic elements required in a document of this nature. Recommendations for enhancing the current plan were made which included measures for earthquake damage detection and isolation, establishment of restoration priorities, stockpiling of repair inventories, coordination of recovery efforts, and establishment of mutual aid agreements.

It is anticipated that the recommendations contained in this report will need to be funded, scheduled and implemented over a 5 to 7 year period of time. In order to assist the SWD to prioritize the necessary upgrades, recommendations have been included in this report as to the order by which the work should be carried out.

2.0 INTRODUCTION

This report presents the results of preliminary seismic evaluations of selected pipelines, structures and equipment located at important facilities owned and operated by the City of Seattle Water Department (SWD). These investigations were conducted by Cygna Consulting Engineers (as prime consultant), along with Shannon & Wilson (geotechnical consultant), Lin & Associates (MBE), and Anne Symonds & Associates (WBE).

The SWD service area and watersheds lie in seismic risk zone 3, in which there is a potential for major earthquakes (7.5 Richter magnitude or larger). The water system is considered a lifeline and must stay operable after a major earthquake to provide fire fighting capabilities, to maintain sanitary conditions and to provide potable drinking water. The SWD system was initiated before the turn of the century with facilities added or replaced up to the present. As a result, many of the facilities are more than 40 years old and were built under less stringent earthquake code requirements than those of today.

Due to the importance of maintaining an operable water system after a major earthquake, the likelihood of having a damaging earthquake, and the age of most of the SWD facilities, the SWD hired Cygna to assess the reliability of their water system. This assessment included the review and verification of seismic ground motion information developed by a previous consultant for SWD's service area and evaluation of potential geologic hazards at particular facilities and along major pipelines. Cygna, and their consultants, then used this information to assess the vulnerability of the SWD facilities and pipelines to earthquakes and also performed detailed seismic analyses on designated facilities, such as elevated water tanks and standpipes. Recommendations were then made concerning the necessary steps SWD should take to improve the seismic reliability of their facilities and pipelines. The existing SWD emergency response plan was also reviewed with regard to earthquake preparedness considerations.

The scope of work for this seismic evaluation project, the results of which are described herein, consisted of the following key items:

1. Verification and Determination of Seismic Hazards in Puget Sound Region

a) A review and verification of existing seismic ground motion information developed previously was performed, including but not limited to information developed by Converse Consultants for SWD and presented in their report on the seismic stability of SWD's dams and reservoir embankments. This information included estimated horizontal ground motion parameters for two levels of design earthquakes: 1) Class I – probable maximum earthquake with low return period and 2) Class II – earthquake with 100 year return period.

b) Seismic criteria were established for use in the facility review that are typically accepted and used throughout the industry (i.e., 500-year return interval for Class I earthquake).

i) Ground motions for Class I and II earthquakes were established based upon seismic studies performed by Shannon & Wilson for numerous sites within the Puget Sound area (including studies at the Tolt and Cedar Rivers).

ii) Response spectra were established for Class I and II earthquakes appropriate for different soil conditions within the distribution network with particular emphasis on the long period contribution of motion within the response spectra.

c) Recent ongoing research on seismic hazards was reviewed as applicable to this region.

d) A seismic hazard map of the Puget Sound area was developed as an overlay to the SWD general arrangement water supply map which delineated potential slide areas and identified seismic zones for three classifications of soil types.

e) A liquefaction hazard map of the Puget Sound area was developed (also as an overlay) which identified areas of historic and potential liquefaction.

2. Facility Seismic Vulnerability Assessment

Site walkdowns and seismic/structural assessments were conducted for the following key SWD facilities and any key equipment items contained therein:

a) Tolt Regulating Basin, Landsburg Diversion and Lake Youngs treatment plants including auxiliary structures required for their functioning, such as screenhouses, water tanks and main valve houses.

b) Nine water storage standpipes and seven elevated steel water storage tanks.

c) Thirty-eight pump stations and post-chlorination treatment facilities.

d) Major pipelines, which consist of 150 miles of supply lines and about 75 miles of major mains including tunnels and intakes. There are also a number of underwater crossings which were evaluated, such as at the Duwamish River, the Ship Canal and the East Channel of Lake Washington.

e) Important valve structures and controls on all major pipelines. This included air vent stacks and vulnerable points such as blowoffs and air valves.

f) Water Operations Control Center and adjacent structures.

g) Control Works Buildings.

Field and site reconnaissance studies were conducted at each facility site to review any site-specific geologic hazards that could affect the post-earthquake performance of the facility. Additionally, site reconnaissance surveys were conducted at specific points along those pipelines which were determined to be in high geologic-hazard areas. The site surveys reviewed the topography and geology within the project areas to evaluate if additional geologic hazards existed at these individual sites.

Construction records of tanks situated on unknown soils were also reviewed to determine whether the footings were located on native soils and fill.

3. **Emergency Response Plan**

The pertinent elements of SWD's existing emergency response plan were reviewed with respect to a major, damaging earthquake, for appropriateness and completeness. Cygna recommended modifications and improvements to the plan in terms of both response and immediate recovery planning measures.

4. **Recommendations for Existing Facility and Pipeline Upgrade**

Possible alternative steps that could be taken to improve the seismic reliability of those facilities or pipelines which were found to be vulnerable are presented at the conclusion of this study. Each specified recommendation is accompanied by a cost estimate for SWD budget funding purposes.

3.0 EVALUATION METHODOLOGY

The seismic assessment of the inventoried facilities and pipelines due to both the Class I and Class II earthquakes was performed considering both structural and nonstructural components and equipment. The assessment consisted of 1) data reviews, 2) site visits, and 3) preliminary seismic evaluations.

The data reviews included a review of facility drawings and specifications, seismic calculations when available, material test reports, soil reports, maintenance reports, inspection reports, and past earthquake performance information. The site visits identified and evaluated the configuration, condition, workmanship, connections and details, existing dead and live loads, etc., of both structural and nonstructural items. Special attention was paid to telemetry capability and equipment to ensure that critical functions would be maintained during an earthquake. The preliminary seismic evaluations of structural and nonstructural components were based on experience with similar structures, nonstructural components and equipment to resist lateral loads during past earthquakes. This experience-based determination was supplemented with hand calculations, using current editions of the applicable codes, as appropriate.

Checklists were used during the field investigations to facilitate the investigations. Figure 3.0-1 shows a typical checklist used for building structures and Figure 3.0-2 shows a checklist used for equipment and non-structural items.

Building structures were generally evaluated using guidelines suggested in the "ATC-14 – Evaluating the Seismic Resistance of Existing Buildings." The evaluation consisted of a site visit, review of existing drawings and calculations as required to investigate potential problem items. The seismic forces used in the evaluation were those resulting from the Class I and Class II earthquakes.

The seismic performance of the major pipelines was evaluated for major geologic hazards, such as landslides or liquefaction, that could affect the earthquake performance of the system. Areas where such hazards may occur were determined from the seismic hazard map and from SWD's performance and maintenance records.

Seismic response spectrum analyses were conducted for the nine water storage standpipes and seven elevated steel water storage tanks and for the Control Works building. These analyses used appropriately developed mathematical models in conjunction with the ANSYS computer code. All these structures were analyzed for the Class I earthquake. Those structures which were found to be not operable after a Class I earthquake were then analyzed for the Class II earthquake. The criteria used for non-failure was that some damage could be sustained provided that the facility was still operable following a Class I earthquake, and that minor structural damage was allowed after a Class II event.

In estimating the operability of the water storage tanks, elastic analyses were performed without modifying the response spectra to account for ductility. This approach was selected because the tanks being evaluated have a combination of brittle and ductile elements and/or modes of failure. Results were interpreted by evaluating the stress at the element level. In this manner, specific characteristics of each structure were accounted for. For example, overstress of a diagonal brace in tension would not be a concern unless the overstress exceeded 160%, which corresponds to a ductility of 4. On the other hand, foundation uplift would have an immediate detrimental effect on the stability of the structure. It is important to note that when the limiting element is a ductile element, such as a diagonal brace in tension, the forces that can be introduced in the structure are limited. In such cases, the elastic analyses overestimate the stresses in the structure and the results were factored accordingly.

An important consequence of this limiting effect is that marginal elements (i.e., foundation uplift) might not be overstressed if a ductile element (i.e., diagonal tension brace) limits the load transferred to it. If the brace were to be strengthened to prevent it from yielding, other elements would undergo increased stress.

In the computer analysis of the elevated storage tanks and standpipes, horizontal and vertical seismic loads were considered. Vertical seismic loads were assumed to be two thirds of the horizontal. This assumption is conservative, as historical earthquakes in the Pacific Northwest have shown the vertical seismic component to be actually closer to one third of the horizontal. Vertical seismic loads were applied both upward and downward, searching for the critical condition for each structural component. Steel and concrete elements were evaluated for their ultimate capacities in accordance with AISC and ACI codes. Hydrodynamic effects were calculated using Housner's methodology.

Using Housner's methodology (See Figure 3.0-3 for a sketch of the mathematical model), the mass of the contained liquid (M) is replaced with a mass M_0 rigidly attached to the tank at an elevation H_0 above the bottom, plus a mass M_1 attached through springs of total stiffness K at elevation H_1. These parameters are given by

$$M_0 = \frac{\tanh 1.7R/H}{1.7R/H} M$$

$$M_1 = \frac{0.71 \tanh 1.8H/R}{1.8H/R} M$$

$$H_0 = 0.38H \left[1 + \alpha \left((M/M_0) - 1 \right) \right]$$

$$H_1 = H \left[1 - 0.21 (M/M_1)(R/H)^2 + 0.55\beta (R/H) \left(0.15 (RM/HM_1)^2 - 1 \right)^{1/2} \right]$$

and

$$K = \frac{4.75g\ M_1^2 H}{MR^2}$$

where g is the acceleration of gravity, R is the radius of the cylindrical tank and H is the depth at rest of the contained liquid. In the case of non-cylindrical tanks, R is approximated as the mean radius of the free surface.

In general, the contribution of the sloshing water to the response of the system was found to be small, the reasons being the low spectral acceleration associated with the long period of sloshing response, and in the case of tall cylindrical containers, the relatively small mass of sloshing water. Accordingly, as a conservative approximation, the height of the masses M_0 and M_1 was taken to be the height of the centroid of the tanks. This height was calculated by including the weight of both the water and the steel.

The three-dimensional computer models of the supporting structural members and foundations included consideration for the columns, beams, braces, and the equivalent radial and tangential beams and the riser in order to represent the stiffening effect of the tank shell and the tank-specific configuration. Experience has shown that the tank shell of elevated tanks is not a critical element in the event of seismic loads. This fact was confirmed through parametric studies which were conducted for the Beverly elevated tank shell, which showed the shell was stressed to only 46% of its allowable. Tank shells are conservatively designed to withstand hydrostatic loads, and during an earthquake, the water level only fluctuates $\pm$ 2 to 3 feet which has only a small, increased effect on the shell stresses. Accordingly, the mathematical modeling of the elevated tanks did not include a refined model of the tank shell.

The fundamental period of the structure is defined as the period with the highest mass participation factor. In typical cantilever structures, this period happens to be the largest natural period. In the case of water storage structures, the largest period is that of the sloshing water mass, and is different from the fundamental structural period.

The elevated tanks have their fundamental period in the region of the descending branch of the seismic response spectra. Standpipes, on the other hand, have the fundamental period in the region of the ascending branch of the seismic response spectra. Special care is taken to avoid underestimating the flexibility of the standpipes because to do so would result in underestimating the seismic load in the standpipe. The effect of soil flexibility was introduced in the computer models using Newmark and Rosenblueth formulations for rocking and vertical stiffness (K_θ, K_v) as follows:

$$K_0 = 2.7\mu r^3$$

$$K_v = \frac{4\mu r}{1-\nu}$$

where ν is Poisson's ratio, μ is the modulus of rigidity, and r is the radius of the foundation. The shear modulus of the soil was assumed to be the average of the small and large strain values: $\mu = 25$ ksi. In each of the elevated tank and standpipe sections of this report, the soil stress values reported in the stress ratio summary tables are based on the allowable rather than ultimate soil capacity.

The tank shell buckling potential was evaluated in accordance with API 650. The geometry has to satisfy the following minimum requirement:

$$GHD^2/t^2 < 10^6$$

and the allowable stress is

$$F_a = \frac{10^6\ t}{2.5D} + 600\sqrt{GH}$$

where:

t is the shell plate thickness in inches; H is the height of liquid from the top to the shell course being considered in feet; D is the tank diameter in feet and G is the liquid specific gravity (1.0 for water).

Stress ratios against a representative failure or yield parameter were evaluated for the different structural elements and different loading conditions. Stress ratios for soil in compression were based on allowable levels.

A summary of the analysis results is presented within the report section for each structure. The summary includes a bottom-line operability statement and an estimated probability of failure for the structure subjected to the Class I earthquake. The estimated probability of failure represents a "best judgment" and is not an explicit result of calculations performed because the seismic spectra inputs used in this work were developed on a deterministic basis.

The base shear resulting from the application of the Class I and Class II earthquakes was compared to the design base shear computed using the 1988 UBC code.

For the Class I and Class II earthquakes, the base shear was obtained from the ANSYS analyses. The UBC sections applicable to elevated tanks and standpipes are Sections 2312(i) 3 and 4. The section applicable to buildings is 2312(e) as follows:

for non-rigid structures ($T \geq 0.06$ seconds)

$$V = \frac{Z\,I\,C}{R_w}\,W \qquad \text{(UBC Section 2312(e)4)}$$

where

$$C = \frac{1.25S}{T^{2/3}} \leq 2.75 \qquad \text{(UBC Section 2312 (e))}$$

$Z = 0.3$ (Seattle is Zone 3, UBC Table 23-I)

$I = 1.25$ (Essential Facilities, UBC Table 23-L)

$R_w = 3$ (Elevated Tanks, UBC Table 23-Q)

$R_w = 4$ (Standpipes, UBC Table 23-Q)

$R_w = 6$ (Concrete/Masonry Shear Walls, UBC Table 23-O)

$S = 1.2$ (firm silty sand, UBC Table 23-J)

and

$$\frac{C}{R_w} \geq 0.5$$

The base shears for standpipes, which are laterally supported by the building (e.g., Queen Anne No. 1, Volunteer Park and Control Works), are calculated per Section 2312(g):

$$V = ZI\,C_p\,W_p$$

where:

$C_p = 2.0$ (Tanks, UBC Table 23-P)

BUILDING DATA Site No. _____ Date: ________ Building Plan:
Site: __________ Structure: ________________
Year built: _____
Area, sf: ______ Length ______ Width _____
No. stories _____ Story height ___ Total Ht. ___

CONSTRUCTION DATA

Gravity load structural system: ____________________
Exterior transverse walls: ______________ Openings? ______________
Exterior longitudinal walls: ______________ Openings? ______________
Roof materials/framing: ____________________
Intermediate floors/framing: ____________________
Ground floor: ____________________
Columns: ______________ Foundation: ______________

LATERAL FORCE RESISTING SYSTEM

	Longitudinal	Transverse
Diaphragms:		
Vertical Elements:		
Connections:		
Details:		

General condition of structure? ____________ Evidence of settling? ______________
Special features/comments:

EARTHQUAKE DAMAGE POTENTIAL*

	Nonstructural	Structural
Level 1 Earthquake:		
Level 2 Earthquake:		

Effective earthquake acceleration: Level 1 ____ Level 2 _____

* see attached note for potential damage states

Typical Building Structure Field Checklist

Figure 3.0-1

Potential Damage States:

A. Nonstructural

- Negligible damage to nonstructural components. Perceptible but generally cosmetic in nature. No repairs required.

- Minor nonstructural damage. A few walls and partitions cracked; incidental mechanical and electrical damage. Repairs while occupied.

- Moderate nonstructural damage. More extensive cracking but still not widespread; can be repaired; possible damage to elevators and other mechanical/electrical components. Repairs required prior to occupation.

- Major nonstructural damage. Widespread cracking to architectural and other nonstructural components rendering the facility to be non-operational; major costs involved for repair or replacement; in some cases repair is not feasible and replacement is necessary.

B. Structural:

- Negligible structural damage. Threshold of cracking or yielding may be attained in a few structural members. No repairs required.

- Minor structural damage. Visible cracking or yielding in a few structural members. Repairs while occupied.

- Moderate structural damage. Repair or replacement required for some structural members. Repairs required prior to occupation.

- Major structural damage. Widespread damage requiring repair or replacement of many structural members. Demolition.

Figure 3.0-1 (Continued)

Site No. ________ Date: ____________
Site: __________________ Structure: ______________

EQUIPMENT DATA

Equipment Item No. ____ Location in Building ______________________________
Description: ____
Sketch: Height: ________
Length: ________
Width : ________

Connecting piping and conduit: ______________________________

Other attachments: ______________________________________

Points of anchorage: ________ Type/size of anchors: ___________
(show on sketch)

Cabinet Framing: ____

EARTHQUAKE DAMAGE POTENTIAL (Y/N)

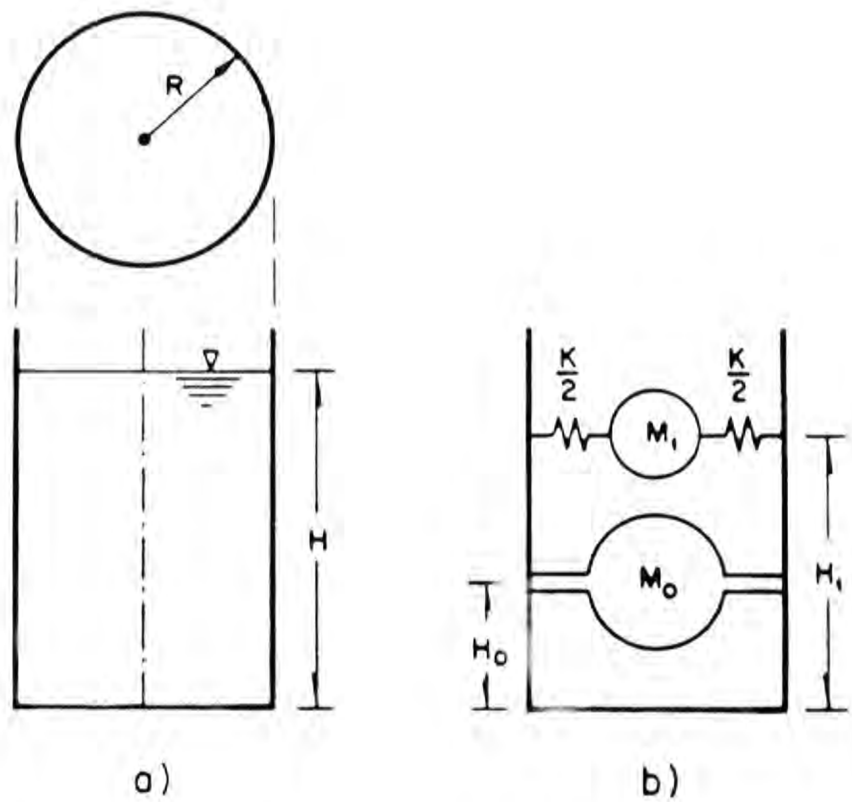
	Class 1	Class 2
Anchor pull out	_____	_____
Anchor failure	_____	_____
Frame damage	_____	_____
Overturning	_____	_____
Sliding	_____	_____
Impact by neighboring item	_____	_____
Rupture of attached piping or conduit	_____	_____
Other: ____________________	_____	_____

Effective earthquake acceleration: Class 1 ____ Class 2 ____

Special features/comments:

Typical Equipment Field Checklist

Figure 3.0-2



Note: For the coefficients α and β, as shown in the equations on sheet 3-2, $\alpha = 1.33$ and $\beta = 2.0$, if the hydrodynamic moment on the tank bottom is to be included in the computations, while $\alpha = 0$ and $\beta = 1$ if only the effects of hydrodynamic pressures on the container walls are of interest.

Model of Circular Cylindrical Tank and Equivalent Masses

Figure 3.0-3

4.0 GEOTECHNICAL REVIEW

A geotechnical assessment was conducted for the Seattle Water Department facilities for the following purposes:

- To establish earthquake parameters appropriate for use in the structural evaluation of the system components.
- To assess the geologic hazards that may affect the operations of the system following a major earthquake.

Several steps were undertaken to characterize the engineering parameters of the earthquakes selected for the review of the facilities. First, earthquake studies that were previously performed for the Seattle Water Department were reviewed to determine the applicability of the recommendations for the review of the various components within the system. Next, in conjunction with the Seattle Water Department, seismic criteria were established for the review of the facilities. The seismic criteria review established the following design earthquakes: a Class I event, corresponding to an earthquake having a relatively low probability of occurrence; and a Class II event, corresponding to an event which is likely to occur during the operational life of the system. Finally, a seismic hazard map was developed illustrating the potential geologic hazards throughout the SWD system. Earthquake ground motions were developed corresponding to the different geologic units identified on the seismic hazard map for the two earthquake levels. Additionally, response spectra were developed for the same geologic units and earthquake levels.

The second major aspect of the geotechnical review consisted of conducting geologic reconnaissances for the major facilities and components of the SWD system. The purpose of these field reviews was to observe the existing condition and performance of the facilities and to document any potential geologic hazards, such as landslides or liquefaction, that could affect the system operation. The findings of these site reconnaissances are summarized in the geologic hazard assessment for the major facilities in the following sections of the report.

4.1 Review of SWD Reservoir Study

One of the first tasks of the geotechnical assessment was to review existing earthquake studies that have been performed for the SWD to determine if the recommendations contained in these reports are appropriate for the seismic evaluation of the overall system. The most relevant earlier study reviewed was a report prepared by Converse Ward Davis Dixon (1980) which specified earthquake motions for evaluating several reservoirs operated by the SWD. This earlier study established review criteria for two earthquakes: a Class I event, having a return interval of 10,000 years and a Class II event, having a 100-year return interval.

The recommended ground motions and response spectra contained in the Converse Ward Davis Dixon (1980) report appear reasonable and conservative for use in the study of the reservoirs. Because these ground motions and response spectra were developed for sites underlain by similar geologic

conditions (glacially consolidated sediments), these recommendations may not be entirely applicable for evaluation of the overall system facilities and components, which are underlain by diverse subsurface conditions which range from alluvial materials and fill soils to competent bedrock. Therefore, additional studies were undertaken to develop ground motion parameters and response spectra for the diverse soil conditions found throughout the SWD system. These recommended ground motions and response spectra are discussed in detail in Section 4.3 of this report.

4.2 Seismic Criteria Development

One of the most important items of the seismic vulnerability assessment is establishing appropriate criteria for the Class I earthquake. Selection of too stringent criteria (high earthquake return interval) would impose unnecessary economic expenditures on system rehabilitation and design; conversely, selection of too lenient criteria (low earthquake return interval) places the general public at an undue risk during a major seismic event. Therefore, it is necessary to establish an appropriate return interval for the Class I event which is also consistent with criteria that have been used by other agencies in the design or evaluation of various lifeline systems.

Earthquakes with various return intervals have been used for the seismic evaluation of lifeline facilities in the Pacific Northwest. A 10,000-year earthquake return interval had been selected by Conserve Ward Davis Dixon (1980) for the seismic evaluation of various reservoirs within the SWD system. More commonly, however, earthquakes with a return interval of 500 years have been frequently used by various agencies for the review of lifeline systems. An earthquake with a 500-year return interval was used by the Portland Water Bureau in the evaluation of a major elevated water tank within their system (Anoushiravani, et al., 1986). Similarly, an earthquake having a 500-year recurrence interval was used to evaluate the seismic stability of the South Fork Tolt River dam for the SWD (Shannon & Wilson, 1981). The 500-year earthquake selected for the Tolt study was also considered to be equivalent to a Maximum Credible Event (MCE). A 500-year earthquake recurrence interval has also been recommended by the Applied Technology Council (ATC, 1978) for the seismic design of bridges and buildings.

Therefore, based upon past precedence, it is recommended that a 500-year recurrence interval be adopted for defining the Class I event. There would be approximately a 10 percent chance that this earthquake would be exceeded at least once during a 50-year period. This represents a reasonable design criteria which is consistent with current local practice.

4.3 Seismic Hazard Map

4.3.1 Approach

A microzonation map was prepared for the purpose of delineating geologic hazards and identifying zones with different ground shaking potential within the SWD service area. This map, which is presented in Figure 4.3-1, was developed primarily on the basis of subsurface soil conditions. It was considered that relatively favorable seismic performance would be provided by sites that are underlain by stable soils, whereas, more adverse seismic performance would be experienced at sites underlain by less competent soils or unstable ground. Accordingly, a system was devised containing three seismic hazard zones corresponding to the following subsurface soil conditions: Zone I - bedrock, Zone II - glacially consolidated sediments, and Zone III - alluvium and landslide potential areas. The seismic hazard map was prepared based upon existing geologic maps of the area (Waldron and others, 1962; Waldron, 1967) and supplemented with existing maps which illustrate the landslide potential in Seattle (Tubbs, 1974) and landslide hazard maps which are on file with the City of Seattle.

The seismic hazard map presented in Figure 4.3-1 was originally prepared at a scale of 1 inch equals 1 mile to correspond to the SWD Facilities Map. A mylar copy of this full-scale map has been provided to the SWD for use as an overlay on their existing Facilities Map.

The purpose of developing the seismic hazard map as an overlay to the existing Facilities Map was to provide a means of identifying components of the system that are located in areas susceptible to geologic hazards (i.e., landsliding or liquefaction). The overlay would also facilitate identifying appropriate levels of ground motion to be used in the structural evaluations of the facilities. It is intended that this information would be used by personnel from the water department to assess areas that may be susceptible to line breaks or leaks during a future earthquake. This information would aid the water department in developing sufficient stockpiles of material as part of an earthquake response plan.

4.3.2 Geologic Hazards

4.3.2.1 Faults

Ground displacements associated with faulting have historically been a major threat to lifeline facilities and have resulted in ruptured pipelines or collapsed buildings. Information on known faults within the SWD service area has been gathered and illustrated on Figure 4.3-1.

Based upon our review of the geologic data, it is concluded that surface faulting will not be a major concern to the operation of the SWD system for the following reasons: 1) none of the known fault structures within the SWD service area appeared to have been active during the past 10,000 years; 2) earthquake activity within the Puget Sound area has generally not been correlated with any known or inferred tectonic structures; and 3) most of the service area is underlain by 1,000 to 3,000 feet of glacial sediments which are unlikely to experience rupture in the event of any fault displacement.

UNDERLYING

4.3.2.2 Landslides

Landslides may have a significant effect upon the SWD facilities in that these mechanisms may sever underground pipelines or undermine support from above ground facilities. Landslides may be induced from seismic activity or, more commonly in the Puget Sound area, from increased groundwater levels during the winter.

Areas having a significant potential for the development of landslides are indicated on Figure 4.3-1. Zones having a high landslide potential have been identified based upon geologic data (Tubbs, 1974) and data from the files of the City of Seattle and other municipalities that indicate sites of historic landslide activity.

This landslide hazard information may be used to identify areas in which pipelines may be damaged following an earthquake. A sufficient stockpile of replacement pipe should be maintained to repair potential leaks in these areas. Finally, this landslide hazard information may be used for planning new service lines, to avoid potentially unstable ground.

4.3.2.3 Liquefaction

A liquefaction hazard map, as shown in Figure 4.3-2, was also prepared to delineate potentially hazardous zones within the service area. The subsurface soils identified within the system area as having the greatest potential for liquefaction are the Holocene alluvial deposits that are found along the low-lying floodplain areas. These alluvial soils consist of sands and silty sands that have relatively low Standard Penetration Resistance values. Liquefaction has been observed within these soils during the two largest historic earthquakes in the region: the magnitude 7.1, April 13, 1949, Olympia earthquake and the magnitude 6.5, April 29, 1965 Seattle-Tacoma earthquake. The locations of these observed areas of liquefaction are indicated on Figure 4.3-2.

Areas having a very high damage potential from liquefaction are also indicated on Figure 4.3-2. These areas are primarily located within 200 feet of streams or open bodies of water. These areas are particularly hazardous as liquefaction beneath the banks of rivers could result in the development of lateral spreads or zones of horizontal soil movement. Pipelines or other structures located within these zones are at a significant risk of sustaining catastrophic damage from a large earthquake. It is noted from Figure 4.3-2 that the majority of historical sites of liquefaction within Seattle are located within this potentially high hazardous zone. By identifying these potentially high hazardous areas, appropriate planning measures may be undertaken to both stockpile replacement pipe for emergency response purposes, as well as to provide for isolation valves on either side of the affected zones.

While liquefaction could also occur in the area located outside the 200-foot limit, liquefaction in these interior areas is not as potentially damaging, due to the reduced potential for lateral movement. Liquefaction in these interior areas may result in some local vertical movement of pipes or settlement of building footings that are not supported on piling.

Underground water lines within the SWD system sustained minor damage following the 1949 and 1965 earthquakes. This damage was locally scattered throughout the city. The majority of this damage, however, was from leaks in mains or services (as opposed to breaks) located along the low-lying areas of the Duwamish. These areas have been identified in Figure 4.3-2 as being potentially liquefiable. Similar damage patterns, with leaks and breaks concentrated in areas identified as having a moderate to high potential for liquefaction, are expected to result from future earthquakes.

4.3.2.4 Seiches

A seiche is an oscillation or wave at the surface of an enclosed body of water that can be generated by a distant earthquake. Factors affecting seiche generation include earthquake size and location, travel paths of seismic energy, regional tectonic structural characteristics, local subsurface geology, and characteristics of the basin geometry. The principal hazard from aseismically-induced seiche is from flooding or wave runup along the shoreline.

Some aspects of the geology in the Puget Sound area tend to decrease the opportunity for seiche generation. Thick, glacially-overridden soils, which are present in much of the region, are thought to be rigid enough to attenuate horizontal seismic waves which generate seiches. Many water bodies, including the Puget Sound, Lake Washington, Lake Sammamish, Lake Youngs, etc., are situated predominantly in these rigid sediments; Chester Morse and Tolt reservoirs are predominantly in rigid bedrock.

Tectonic structures and trends in the region, however, may somewhat increase the likelihood of seismic seiche generation, especially if the direction of wave propagation is north-south. The north-south trending Cascade range is theorized to channel seismic energy along its length. This channeling effect could cause amplification of seismic surface waves and increase the possibility of seismic seiche generation, especially in bodies of water close to the range, such as Chester Morse and Tolt reservoirs. The Puget Sound region sits in a north-south trending tectonic trough. Tectonic troughs are also thought to concentrate seismic energy. The increased amplification that would occur to the seismic surface wave would also increase the opportunity for seiche generation for water bodies in the trough, such as the Puget Sound, Lake Washington, Lake Union, and Lake Sammamish.

Because the geometry of a water body also affects seiche development, the geometry of large reservoirs and deep bodies of water tends to be more conducive to the generation of seismic seiches than do rivers and streams. Bodies of water in the region that may have geometries conducive to seiche generation include the Puget Sound, Lake Washington, and possibly the Chester Morse and Tolt reservoirs.

Although historical seiches are rare in Washington, they provide the most useful data relative to the seiche hazard in the area. Runups of as much as 8 feet have been reported on Lake Washington. The Puget Sound has recorded seismic seiches up to 0.40 feet, double-amplitude; where double-amplitude refers to the distance between the runup height and the recessional trough. For reservoirs in Washington state, 1.04 feet, double-amplitude seiches have been recorded. In our opinion, these runup heights should be used as a basis for estimating the potential effects for seiches upon the components of the SWD system.

4.3.3 Ground Motions

Seismicity studies performed by Shannon & Wilson for numerous projects in the Puget Sound area were used as a basis for estimating ground motions appropriate for evaluating the performance of the various components within the SWD system. Based upon these studies, it was considered that the most likely source mechanism for the Class I earthquake would be a deep, subcrustal event. This type of earthquake is not associated with known tectonic structures. Therefore, this type of earthquake could occur anywhere within the Puget Sound lowland. However, it is less likely that this event would occur along the margin of the Cascade physiographic province, in the vicinity of the Tolt Reservoir, or the facilities at Lake Youngs.

Thus, based upon the concept that the controlling Class I earthquake could essentially occur anywhere within the service area, equal probabilities of earthquake occurrence were assigned throughout the service area and different levels of ground shaking were differentiated based strictly upon subsurface conditions as defined by the seismic zones in Figure 4.3-1. The assumption of uniform ground shaking potential throughout the region is particularly conservative for the outlying facilities, such as Tolt and Lake Youngs, as it is less likely that the design earthquake would occur in this area, as compared with areas more centrally located in the Puget Sound trough.

Therefore, the following peak ground acceleration values are recommended for each of the three seismic zones identified on Figure 4.3-1 and the two design earthquake levels:

Peak Ground Acceleration (g)

Earthquake	Zone I (Bedrock)	Zone II (Glacial Sediments)	Zone III (Alluvium)
Class I (500-yr.)	0.24	0.27	0.32
Class II (100-yr.)	0.15	0.17	0.20

It is estimated that the Class I event would be correlated with an earthquake of Richter magnitude 7.5, whereas the Class II event would be correlated with a magnitude 6.5 earthquake.

The above table indicates that ground motions on soil deposits will be amplified as compared to ground motions at bedrock sites. The concept of amplification of ground motions for soil sites has been widely debated within the scientific community. Specifically, relations developed by Seed would suggest that peak ground

accelerations in soil deposits would be less than those at comparable rock sites. Other empirical relationships developed by Trifunac and Brady would suggest that there is no difference in ground accelerations recorded at either soil sites or rock sites. Finally, Nuttli has suggested that soil sites would tend to amplify peak ground motions. Thus, review of the technical literature indicates that there is no consistent agreement upon the relative importance of subsurface conditions upon recorded peak ground motions. However, based upon Shannon & Wilson's experience in reviewing historical damage reports from earthquakes in the Puget Sound area, it has been concluded that most structural damage has been sustained on sites located on alluvial-type soils. This damage reflects not only the vibratory ground shaking effects, but also the potential effects of foundation settlement. Therefore, the alluvium soils within Zone III were selected as having the highest relative seismic hazard of the three material types. Conversely, it was considered that sites underlain by bedrock (Zone I) would provide the best foundation performance.

The above recommended ground motions for the Class I earthquake agree favorably with the results of probabilistic studies performed by others. Specifically, the recommended 0.24g ground acceleration for sites underlain by bedrock (Zone I) is in reasonable agreement with the results of probabilistic studies performed by the U.S. Geological Survey (Algermissen, et al., 1982; and Algermissen, 1988). Similarly, the 0.27 and 0.32g peak ground accelerations for the glacial sediments and alluvial deposits agree favorably with peak ground motion values inferred for seismic Zone 3 from the Uniform Building Code. Therefore, from the above comparison, it is concluded that the recommended ground motions are reasonable and constitute a valid basis for evaluating the seismic performance of the components of the SWD.

4.3.4 Response Spectra

Response spectra recommended for evaluating the components within the SWD system are presented in Figure 4.3-3. These elastic response spectra have been prepared for damping levels of 0.5, 3, 5, and 7 percent damping.

The spectra presented in Figure 4.3-3 have been developed based upon site matching studies that Shannon & Wilson has performed for various projects in the Puget Sound region. These response spectra were specifically developed for sites underlain by alluvial soils. However, it is our opinion that these response spectra would conservatively represent ground motions for sites underlain by glacially consolidated sediments or rock. Thus, the same spectral shape, but with different ordinates depending on subsurface geology, may be applied to the study of the SWD system.

The response spectra presented in Figure 4.3-3 represent the concept that the same spectral shape may be applied to all seismic zones; however, the ordinates of the spectra would be modified by the peak ground accelerations for the underlying geologic unit. This concept is analogous to the response spectra recommendations of the ATC (1978) and the Uniform Building Code. These codes specify use of one value of ground acceleration, irrespective of subsurface soil conditions. The response spectra, however, are modified beyond a period of approximately 0.5 seconds depending upon the local site geology. The code spectra for rock sites are lower than those for soil sites, similar to the recommendations provided for ground accelerations for the Seismic Hazard Map. Therefore, it is viewed that the overall concept of the recommended ground motions and response spectra for the SWD study are generally consistent with the recommendations of the ATC (1978) and the Uniform Building Code.

4.3.5 Earthquake Research

As previously mentioned, the results of seismicity studies conducted by Shannon & Wilson for various sites within the Puget Sound area indicate that a controlling event for the Class I earthquake would be a deep, sub-crustal earthquake. There has been, however, recent interest expressed in the scientific community about the potential of an earthquake occurring along the Cascadia subduction zone. The following addresses the relative potential of the occurrence of a Cascadia subduction zone event and its potential implications on the ground motions and response spectra recommended for the SWD facility evaluation.

The concept for the potential occurrence of a Cascadia subduction zone earthquake was initially developed from work by Heaton and Kanamori (1984), suggesting that earthquakes with moment magnitudes in excess of 9.0 may occur on the Cascadia subduction zone. To investigate this hypothesis, Atwater (1987) conducted studies for field evidence of subsided salt water tidal marsh areas along the Pacific Coast, as subsidence has been frequently associated with large earthquakes in other subduction zones. In addition to these studies, the U.S. Geological Survey is currently supporting numerous projects investigating the potential of a large earthquake within the Cascadia subduction zone.

Although there is some evidence to suggest the prior occurrence of earthquakes on the Cascadia subduction zone, there are also numerous anomalies in the area that would tend to discount this possibility and indicate that convergence of the Pacific and North American plates is occurring aseismically. These arguments are based on the fact that convergence of lithospheric plates in other parts of the world typically is accompanied by various types of faulting and other seismic activity. Such typical characteristics are noticeably absent along the Cascadia subduction zone. The following

arguments would suggest that movement along the subducting plates may be occurring aseismically:

- Lack of thrust faults near the plate margin
- Lack of instrumental seismicity along the plate interface
- Lack of geological correlations with uplifted Holocene terrace deposits
- Lack of evidence of strong ground motion (liquefaction) in tidal areas that show subsidence

In light of the above issues, it would appear that there is no overwhelming evidence either substantiating or precluding the potential of a Cascadia subduction zone earthquake. Based upon the evidence compiled to date, we tend to favor the position that subduction of the Pacific plate is occurring aseismically. However, because the possibility of coseismic subduction cannot be entirely precluded, we will address the issue of the potential effects associated with a Cascadia subduction zone earthquake.

Evaluation of an earthquake on the Cascadia subduction zone requires information on the size, location, and potential effects of this earthquake in the Puget Sound region. Heaton and Kanamori (1984) have suggested that a maximum earthquake, with moment magnitude in excess of 9.0, may occur on the Cascadia subduction zone. This estimation is based upon correlating the age of the subducting plate and the rate of plate convergence with other areas having seismically active plate convergence. This analogy, however, has not been rigorously tested against potential geometric constraints of the fault zone. In considering geometric constraints of the fault plane, and the segmentation of the Cascadia subduction zone into three distinct and separate areas, it is concluded that the potential fault zone could have a maximum width of 75 km and a maximum rupture length of 250 km (WPPSS, 1988). A fault with this geometry would result in an earthquake with a moment magnitude of 8.25.

The location of the earthquake with respect to the site is also important. Work by Crosson and Owens (1987) suggests that there is a warp or bend in the subducting Pacific plate that is occurring along a line that intersects the Puget Sound area. This warp, in conjunction with a relatively shallow dip angle of the plate in this area, results in the plate being located relatively close to Seattle. Based upon this evidence, a number of researchers studying the potential effects of a Cascadia subduction zone earthquake in the Puget Sound region have used a hypocentral distance of 60 km to represent the separation distance between Seattle and the nearest point on the subducting Pacific plate. In our opinion, however, this does not represent a reasonable assessment for seismicity in the Puget

Sound area, as this position assumes that the warped portion of the subducting plate is capable of producing the same magnitude earthquake as the more planar plate segments located both to the north and to the south. Therefore, we have concluded it is more rational to assume that the maximum earthquake would occur on the plate boundary segments located either north or south of the warp, which would then correspond to a minimum hypocentral distance of approximately 150 km.

The final link in the assessment of the potential effects of a Cascadia subduction zone earthquake is an estimation of the maximum ground motions that could occur in the Puget Sound area corresponding to this event. Using empirical relationships that have been developed for subduction zone earthquakes (WPPSS, 1988; Crouse, et al., 1988), it is estimated that an earthquake characterized by a magnitude 8.25 event occurring at a distance of approximately 150 km from Seattle, could result in a peak ground acceleration of approximately 0.15g at Seattle. Such an acceleration is less than the values that would normally be considered from the occurrence from a deep, subcrustal event in the Puget Sound area. The occurrence of a potential Cascadia subduction zone earthquake, therefore, would not control the facility design in the region. Similar conclusions that the potential occurrence of a subduction zone earthquake would not necessarily control design parameters in the region have been reached by other investigators (Algermissen, 1988; and Ihnem and Hadley, 1987).

Although ground motions from a potential subduction zone earthquake are not expected to exceed the ground motions resulting from a deep, subcrustal earthquake, it is anticipated that the occurrence of a subduction zone earthquake would have a duration twice as long as that of a subcrustal event. It is estimated that a subduction zone earthquake would have a duration of at least 50 seconds. Heaton and Hartzell (1987) have estimated that such an earthquake may have a duration of 2 minutes.

As previously mentioned, a number of researchers have been studying the potential effects of a subduction zone earthquake in the Puget Sound area. The results from these studies have recently been discussed in a USGS-sponsored workshop on earthquake hazards in the Pacific Northwest, held in Portland, Oregon, on March 29-30, 1989. The presentations from this workshop indicated that a potential Cascadia subduction zone earthquake may produce peak accelerations ranging from 0.1 to 0.4g within the Puget Sound area (Summerville, Youngs, and Wang). Despite this large variation in predicted ground motions, it would appear that most of the researchers would favor ground motion estimates between 0.1 and 0.25g for the Puget Sound area. These ground motion estimates, however, are typically based upon the occurrence of a subduction earthquake within 60 km of Seattle. As previously mentioned,

however, it is doubtful that a large subduction earthquake could occur on the warped portion of the subducting plate, which is located close (60 km) to Seattle. Consequently, in our opinion, these ground motion estimates are most likely too conservative.

It is concluded, therefore, that lower levels of acceleration would be consistent with a reasonable assessment of the location of a potential subduction zone earthquake in the Puget Lowland. These lower ground motion estimates would be consistent with the 0.15g acceleration level previously recommended for a subduction zone earthquake.

Response spectra for a subduction zone earthquake have been developed by various investigators and have a different shape than spectra generated by deep, subcrustal events. The spectra from a subduction zone earthquake have higher amplification values for the intermediate to long period of ground motion. However, considering that this higher amplification in the long period range would be offset by the lower values of peak ground motion, it is unlikely that the response spectra from a magnitude 8.25 subduction event 150 km from Seattle would exceed the spectra recommended for the Class I earthquake.

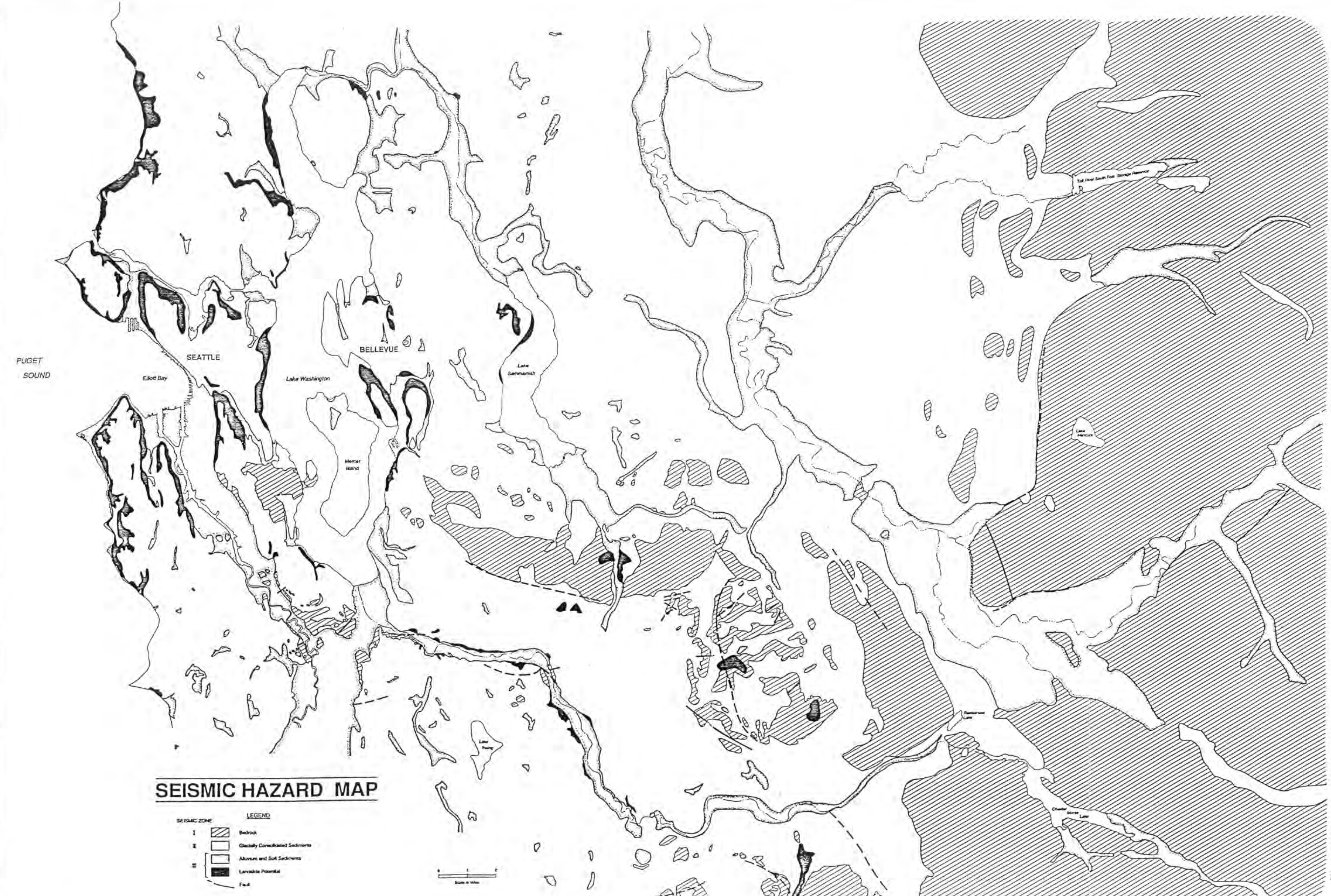


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FIG. 4.3-1

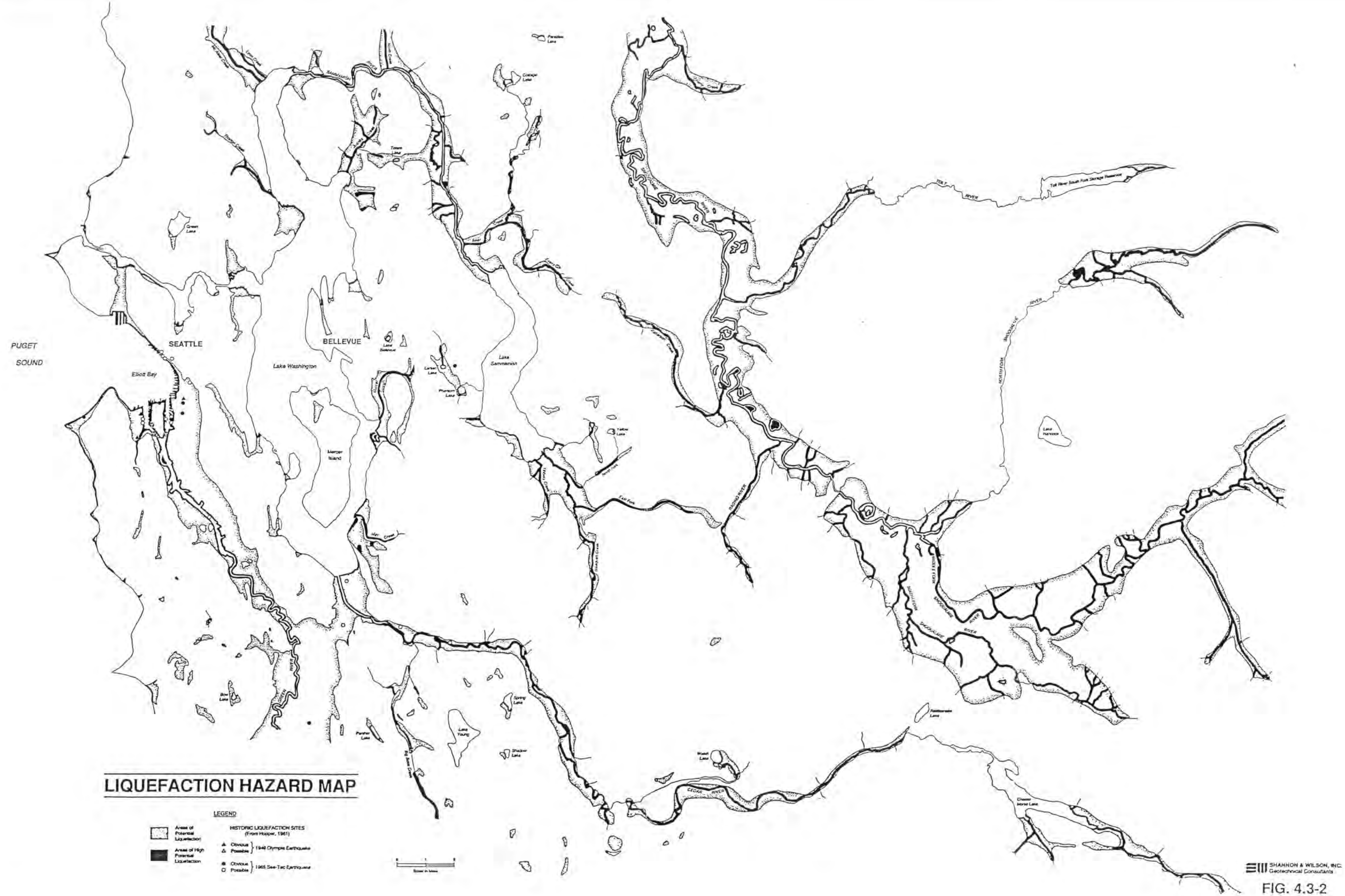
PUGET
SOUND
SEATTLE
Elliott Bay
Lake Washington
BELLEVUE
Mercer Island
Lake Sammamish
Tolt River South Fork Storage Reservoir
TOLT RIVER
NORTH FORK SNOQUALMIE RIVER
MIDDLE FORK SNOQUALMIE RIVER
SOUTH FORK SNOQUALMIE RIVER
CEDAR RIVER
Chester Morse Lake
LIQUEFACTION HAZARD MAP
LEGEND
Areas of Potential Liquefaction
Areas of High Potential Liquefaction
HISTORIC LIQUEFACTION SITES
(From Hopper, 1981)
Obvious
Possible
1949 Olympia Earthquake
Obvious
Possible
1965 Sea-Tac Earthquake
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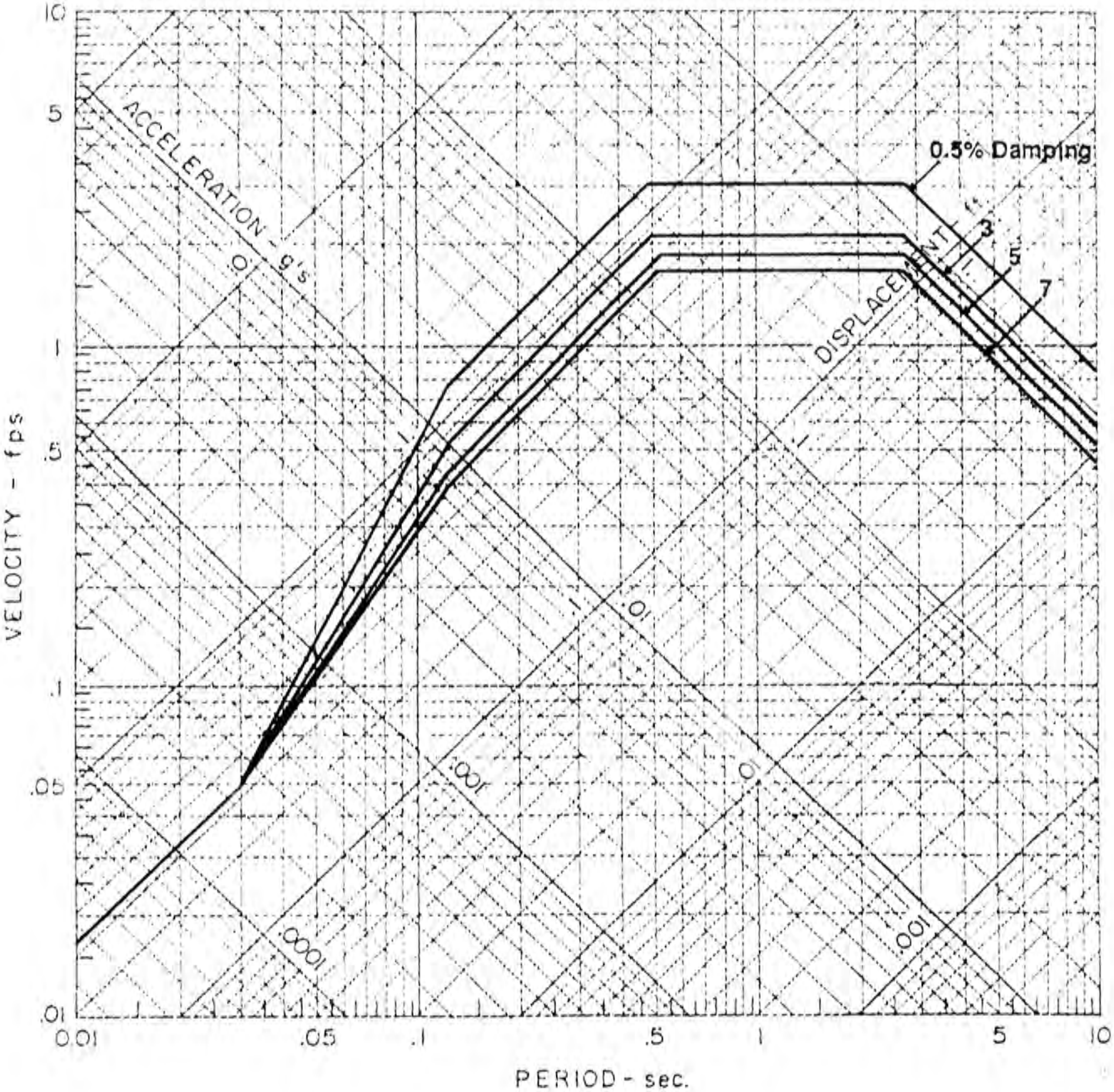
FIG. 4.3-2



<u>NOTES</u>

1. Spectra are for free field, horizontal motions at the ground surface.
2. Vertical spectra correspond to two-thirds of the above values.
3. Spectra are for Class I earthquake and zone III conditions.
4. Use 75% and 85% of the above values for zones I & II, respectively.
5. Spectra for Class II earthquake equal 62% of the above values.

Seattle Water Department Seattle, Washington	
ELASTIC RESPONSE SPECTRA	
May 1989	W-5023-30
SHANNON & WILSON, INC. Geotechnical Consultants	**FIG. 4.3-3**

5.0 TOLT SOUTH FORK FACILITIES

5.1 Facility Description

The Tolt South Fork facilities are essential for the provision of potable water from the Tolt Storage Reservoir. The facilities house the dam outlet control works, and chlorination and corrosion control equipment. There are also maintenance and shop facilities. Disruption of the functions of these facilities would have a significant impact upon the SWD system's capability to deliver water to a large urban area.

5.2 Geologic Hazard Assessment

The majority of the facilities at the Tolt South Fork complex are located upon competent subsurface conditions consisting either of glacially consolidated sediments or bedrock (Zones II and I, respectively, on Figure 4.3-1). Additionally, some of the existing facilities, such as the upper and lower control works, have been constructed upon fill for the South Fork Tolt River Dam. The combination of the generally favorable subsurface conditions beneath these facilities and the fact that most of the facilities are located upon level ground, generally precludes the development of major earthquake-induced geologic hazards such as landslides or liquefaction.

Site reconnaissances that have been performed for the major facilities at Tolt generally indicated that the foundations of the existing structures have generally performed quite well to date with very little signs of distress. Some minor distress, however, was noted in a concrete walkway slab located adjacent to the lower control house. Additionally, it is our understanding that a rock wall adjacent to the screen house experienced some movement during the 1965 Seattle-Tacoma earthquake. Finally, a hairline crack was observed to run through the middle of the fluosilicic acid pad, apparently along a construction joint. In our opinion, none of the above observations constitutes a major threat to the operation of any of the facilities.

The only significant geologic hazard noted during the facility reconnaissance was a small slump located on the hillside slope which is approximately 20 to 25 feet north of the chlorination building. This slump, which is approximately 40 feet wide, appears to be related to heavy groundwater seepage which was observed in a lower portion of the slope. As there is a water tank located on the slope above the chlorination building, it is possible that further slumping of this slope might possibly damage any underground water lines between the tank and the chlorination building. It would appear that the hillside stability could be improved by installing subsurface cutoff drains on the slope to intercept the observed sources of groundwater.

5.3 Duvall Shop

5.3.1 Facility Description

The Duvall shop building serves as a control of operations center for the Tolt South Fork facilities. The metal building, constructed in 1961, is a light steel moment frame building measuring 40 feet by 80 feet. The building eave height is approximately 15 feet. The main function of this building is to shelter and maintain the Tolt South Fork heavy equipment as well as provide a hub for all of the Tolt facilities. The building has two 16 foot wide overhead doors, and one entry door at the west elevation. The building has three transverse rigid frames and two framed end walls. The roof diaphragm consists of "z" purlins spanning between the rigid frames and the framed end walls. This building has typical metal building components, with an added wood framed office portion.

5.3.2 Structure Evaluations

The original construction documents for the metal building show that the office and storage area were originally partitioned by the metal building manufacturer. In 1971, the Duvall Headquarters remodeling added 2"x4" bearing walls supporting a mezzanine framed with 2"x12" at 16 inches on center. Since the framed walls are sheathed with 3/8-inch plywood, we find that this area has a low vulnerability to seismic damage. During the inspection, the metal building was found to be in good condition, including all of the steel bracing and anchor bolts. In general, this type of light steel frame buildings has had excellent performance records in past earthquakes. We rate the structure as having a low vulnerability to seismic damage in a Class I event.

5.3.3 Equipment and Contents Evaluations

There was not much equipment to evaluate; however, the six locker bays on the ground floor need to be bolted down to prevent overturning during an earthquake.

5.3.4 Summary and Conclusions

a) From our preliminary structural investigation, we found that no remedial strengthening of the building is warranted.

b) The locker units, especially those adjacent to the doors in the building, present a seismic hazard from overturning. These lockers should be anchored.

c) Table 5.3-1 summarizes the results of the seismic evaluations. An estimate of remedial upgrade costs is presented in Table 5.3.-2.

Table 5.3-1

Summary of Preliminary Seismic Evaluations
Tolt South Fork Duvall Shop (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Locker Cabinets	2	2	Anchor

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 5.3-2

Cost Estimate

Location: Tolt South Fork Duvall Shop

Date Constructed: 1961

Priority: Low

Seismic Strengthening Objective:

1) Prevent lockers from overturning

Upgrade Recommendations:

1) Anchor six locker bays to floor or tie to wall.

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	600
	Subtotal:	700
4)	Sales Tax (8.1%)	50
	Total:	$ 750

Accuracy of Estimate: ±35%

5.4 Upper Control House

5.4.1 Facility Description

The Upper Control House is located on the west end of the Tolt Storage Reservoir and serves as an auxiliary power outlet valve for the dam. The building, constructed in 1958, is a reinforced concrete structure measuring 11 ft. by 16 ft. with an approximate height of 9.5 ft. above the slab on grade. The building is located adjacent to the Upper Control Bridge.

5.4.2 Structure Evaluations

According to the original construction drawings, the walls are 8-in. inches thick and generally reinforced with #4 bars at 9 in. on center (o.c.) each way, which is adequate reinforcing according to the ATC-14 concrete shear wall checklist. The concrete roof slab is 6-in. thick and is keyed and doweled into the perimeter concrete walls with #4 bars at 18-in. o.c.. Special reinforcing is provided at the window, door, and vent openings. During the site inspection, a few horizontal cracks were observed in the concrete walls above the windows at the south elevation. Further investigation determined that the wall cracks did not propagate from the exterior to the interior. Using the ATC check list, this structure was found to rate very well. Thus, the building structure is judged to have a low vulnerability to seismic damage in a Class I event.

5.4.3 Equipment and Contents Evaluations

The control house building is equipped with an auxiliary generator, a transformer, and gate controls. This equipment was well anchored and has a low vulnerability to seismic damage. The electrical control panels are also properly anchored.

5.4.4 Summary and Conclusions

a) From this preliminary structural investigation, the Upper Control House is adequately designed to resist a Class I earthquake.

b) The equipment within the building has a low vulnerability to seismic damage. No remedial strengthening is required.

c) Table 5.4-1 summarizes the results of the seismic evaluations.

Table 5.4-1

Summary of Preliminary Seismic Evaluation
Tolt South Fork Upper Control House (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

5.5 Upper Control Bridge

5.5.1 Facility Description

The Upper Control Bridge is located at the west end of the Tolt Storage Reservoir and is used to facilitate the control of outflow from the reservoir by changing the elevation of the intake weir. The bridge, constructed in 1958, spans between the spillway gate tower, an intermediate tower, and the reservoir intake tower. The bridge sections are 5-ft. wide by 8-ft. high and vary in length from 72 ft. to 82 ft.

5.5.2 Structure Evaluations

The bridge sections are formed of structural tees for the top and bottom chords with angle web members between. The top and bottom chords are laterally supported by angle bracing and virtually all of the connections are welded. The available details of the bridge tower show that the tower is anchored to a substantial concrete base. The tower is constructed of structural tee members and angle bracing members.

Inspection of the bridge showed that it is in excellent condition. Corrosion of the bridge and other members was not apparent. The bridge structure has a low vulnerability to seismic damage in a Class I event, since the structure is constructed of light steel members that allow yielding and deflection to occur without major damage.

5.5.3 Equipment and Contents Evaluations

The upper control bridge is equipped with gate and valve controls. These controls were anchored adequately with 3/4-in. diameter bolts. The conduit was typically attached to the bridge structure with 1/4-in. diameter bolts.

5.5.4 Summary and Conclusions

a) The Upper Control Bridge is adequately designed to resist the Class I earthquake.

b) The equipment on the bridge structure is properly anchored and has a low vulnerability to seismic damage.

c) Table 5.5-1 summarizes the results of the seismic evaluations.

Table 5.5-1

Summary of Preliminary Seismic Evaluation
Tolt South Fork Upper Control Bridge (High Priority)

Item	Description	Class I	Class II	Comment
1	Bridge	1	1	O.K.
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

5.6 Lower Control House

5.6.1 Facility Description

The Lower Control House is located west of the Tolt Storage Reservoir and the Upper Control House. The Lower Control House contains the outlet valve for the pipeline and a turbine to generate power for use by the SWD. The building, constructed in 1958, is a reinforced concrete structure measuring 7 ft. by 13 ft. for the upper portion, and 16 ft. by 20 ft. for the lower portion. The lower portion is approximately 13 ft. from the floor slab to the roof and is partially underground. The upper portion of the building is about 9-ft. high.

5.6.2 Structure Evaluations

According to the original construction drawings, the upper walls are 8-in. thick and generally reinforced with #4 bars at 9 in. o.c. each way, which is adequate reinforcing according to the ATC-14 concrete shear wall checklist. The concrete roof slab is 6-in. thick and is doweled into the perimeter concrete base formed by the lower structure. Additional reinforcing appears to be provided at window, door, and other openings.

The lower portion of this building has 10-in. thick reinforced concrete walls with reinforcing in each face. The walls are doweled to the foundation and the 10-in. thick concrete roof slab is doweled into the walls. Using the ATC check list, this structure rated very well and has a low vulnerability to seismic damage in a Class I event.

5.6.3 Equipment and Contents Evaluations

The Lower Control House building is equipped with a turbine for generating power. This equipment was well anchored to the slab and is considered to have a low vulnerability to seismic damage. All of the electrical control panels and conduits were properly anchored.

5.6.4 Summary and Conclusions

a) The Lower Control House Structure is properly designed to withstand the Class I earthquake.

b) The equipment within the building is properly anchored and has a low vulnerability to seismic damage.

c) Table 5.6-1 summarizes the results of the seismic evaluations.

Table 5.6-1

Summary of Preliminary Seismic Evaluations
Tolt South Fork Lower Control House (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

5.7 Turbine Building

5.7.1 Facility Description

The Turbine Building is located near the east end of the Tolt Regulating Basin and provides auxiliary power for the Tolt facility. The building, constructed in 1962, is a reinforced concrete structure, 13-ft. wide and 25-ft. long, with a height that varies from about 13 ft. to 9 ft. above the interior slab on grade. The building is located at the end of a very steep slope and the design was site-adapted for that particular location. The wall adjacent to the slope spans horizontally between the return walls and a center concrete column and is 11-in. thick. The north and south elevation walls are also 11-in. thick, and the front wall is 8-in. thick. The roof diaphragm consists of 4 x 6-in. tongue-and-groove (T&G) decking spanning between the front and back concrete walls.

5.7.2 Structure Evaluations

According to the original construction drawings, three of the walls are 11-in. thick and are very well reinforced. There is typically reinforcing in each face of the wall. The 8-in. thick wall at the front elevation is reinforced adequately according to the ATC-14 concrete shear wall checklist. Special reinforcing is provided at window, door, and vent openings. During the site inspection, no wall cracks or signs of settlement were observed.

The roof diaphragm appears to be adequately attached to the rear wall; however, no detail of the connection of the roof diaphragm to the front wall was provided. It appeared from the site inspection that the roof connection at the front elevation may be a weak link. Further investigation should be completed to determine if retrofitting is required. Using the ATC check list, this structure was rated as having a low vulnerability to seismic damage in a Class I event; however, we recommend that the roof diaphragm connections to the front wall be further investigated.

5.7.3 Equipment and Contents Evaluations

The turbine within the building is properly anchored to the concrete pad. The electrical equipment is all bolted to the concrete walls.

5.7.4 Summary and Conclusions

a) The Turbine Building is properly designed to resist the Class I earthquake, however the roof connection to the front wall should be further investigated as a possible "weak link" within this structure.

b) The equipment within the building is properly anchored and has a low vulnerability to seismic damage.

c) Table 5.7-1 summarizes the results of the seismic evaluation. The estimated cost for the additional investigation of the roof-to-wall connection is presented in Table 5.7-2.

Table 5.7-1

Summary of Preliminary Seismic Evaluation
Tolt South Fork Turbine Building (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	Investigate roof connection
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 5.7-2

Cost Estimate

Location: Tolt South Fork Turbine Building

Date Constructed: 1962

Priority: Medium

Additional Investigation Required:

1) Roof-to-wall connection at front elevation

Cost Estimate:

1)	Engineering	$1,600
	Total:	$1,600

Accuracy of Estimate: ±15%

5.8 Screen House

5.8.1 Facility Description

The Screen House is located at the north end of the Tolt Storage Reservoir and serves to screen out the debris from the reservoir. The building, constructed in 1961, is a partially reinforced pumice block structure measuring 25 ft. by 74 ft. The foundation for the screen house building is comprised of 40-ft. deep concrete walls which form the screening chambers. The upper portion of the screen house building is about 11 ft. above this foundation. The roof structure consists of 3-in. and 4-in. wide T & G decking spanning the six glu-lam beams that support the roof structure. These beams span the 25-ft. width of the building and are supported on 16 x 16-in. reinforced block pilasters.

5.8.2 Structure Evaluations

According to the original construction drawings, the block walls are 8-in. thick and are reinforced horizontally with "K-web" joint reinforcing at every third course. No vertical wall reinforcing was specified in the plans except for four #5 vertical bars with #3 ties at each block pilaster. The T & G diaphragm stresses will be low since the beams provide a continuous tie across the building in the transverse direction. The upper walls of this building are tied together with a continuous bond beam through the block pilasters which contains two #5 bars.

The block wall-to-roof diaphragm connection will stress the 4 x 4-in. wood ledger across its grain; however, since the roof beams provide a continuous tie across the diaphragm, this stress will be low and will not pose a problem. A design detail showing the roof connection for the northeast and southwest elevations was not found. The horizontal diaphragm chord steel is several ft. below the actual roof level. This connection should be investigated further to determine if the upper portion of these walls are adequately tied to the roof.

The shear capacity of the structure along its southeast elevation appears to be marginal due to the large amount of window and door penetrations. This should be further investigated through refined calculations to determine if sufficient shear capacity exists. The foundation of the structure is adequately reinforced and is not a major concern. Using the ATC-14 reinforced masonry checklist, this structure has some weak links due mainly to lack of vertical reinforcing. Thus, the structure should be analyzed in more detail to accurately assess its potential seismic risk.

The Screen House structure is rated as having a moderate vulnerability to seismic damage in a Class I event. Other remedial measures may be indicated by the additional studies recommended above;

however, from this preliminary investigation and for the purposes of cost estimation, it appears reasonable to assume that seismic strengthening measures would include: 1) anchoring the roof diaphragm to the end walls; and 2) providing lateral bracing in the southeast elevation wall, including infill of the window penetrations and grouting of the original sill block.

5.8.3 Equipment and Contents Evaluations

Emergency Diesel Generator

The backup emergency diesel generator is located outside at the northeast elevation of the screen house. The generator is covered by a lean-to structure consisting of T & G roof decking extending 6 ft. from the northeast wall of the screen house to a 6-in. thick reinforced block wall. The roof of the lean-to is attached to the screen house block wall using a wood ledger and anchor bolts. This connection is subject to cross grain bending in the wood ledger (which is not allowed by the current Uniform Building Code) during a seismic event. This cross grain bending can be eliminated by bolting a steel strap to the existing anchor bolts and nailing the strap to the roof T & G member above. In addition, the generator should be bolted to the slab.

There is also a primary emergency diesel generator located inside a small room contained within the Screen House. This generator appeared to be properly anchored to withstand a Class I earthquake.

Screen House Equipment

The equipment in the screen house includes switchboards, piezometers and screen motors. All of these items were found to be anchored securely. The wood cabinets located near the northeast elevation wall need to be anchored to the wall.

5.8.4 Summary and Conclusions

a) From this preliminary structural investigation of the Screen House building, several possible weak links have been identified that need to be investigated further. After a more in-depth engineering analysis of some of the items listed in the structure section, some remedial strengthening of the building may be required.

b) The lean-to structure should be strengthened by the addition of steel straps.

c) Most of the equipment within the screen house building has a low vulnerability to seismic damage. The wood cabinets need to be anchored to the block wall for stability.

d) Table 5.8-1 summarizes the results of the seismic evaluations. An estimate of the remedial strengthening costs is presented in Table 5.8.-2.

Table 5.8-1

Summary of Preliminary Seismic Evaluation
Tolt South Fork Screen House Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	More analysis required
2	Lean-to	2	1	Steel straps required
3	Equipment	2	1	Anchor wood cabinets; Bolt generator to slab

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 5.8-2

Cost Estimate

Location: Tolt South Fork Screen House

Date Constructed: 1961

Priority: High

Seismic Strengthening Objectives:

1) Tie roof diaphragm to NE and SW end walls
2) Provide lateral bracing in the southeast elevation wall
3) Secure lean-to roof
4) Anchor diesel generator
5) Prevent wood cabinets from overturning

Upgrade Recommendations:

1) Tie block wall to roof using steel straps
2) Infill southeast wall window penetrations with c.m.u. and grout windowsills
3) Tie lean-to roof to ledger with steel straps
4) Bolt diesel generator to slab
5) Tie wood cabinets to wall

Additional Investigation Required:

1) Verify block wall-to-roof diaphragm connection
2) Determine shear capacity along southeast elevation
3) Evaluate effect that lack of vertical reinforcing has on seismic performance of structure

Assumption: Construction Performed Outside (Except for securing lean-to roof, diesel generator and wood cabinets)

Cost Estimate:

1)	Additional Investigation	$4,000
2)	Engineering	18,000
3)	Construction Engineering	12,900
4)	Construction	27,000
	Subtotal:	61,900
5)	Sales Tax (8.1%)	2,250
	Total:	$64,150

Accuracy of Estimate: ±35%

5.9 Chlorination Building

5.9.1 Facility Description

The Tolt Chlorination Building was constructed in 1961 and treats the water from the Tolt Storage Reservoir. The upper portion of the building is a partially reinforced pumice block structure 41-ft. wide and 60-ft. long. The lower portion of the building consists of a basement measuring approximately 25 ft. by 60 ft. with doubly reinforced concrete walls. The upper portion of the building is about 10 ft. above the floor slab and slopes towards the center of the building, which has about a 9-ft. height. The roof structure consists of 3-in. T & G decking spanning between the six glu-lam beams that support the roof structure. These beams span the 41-ft. width of the building and are supported on four 16 x 16-in. reinforced block pilasters for each beam line.

5.9.2 Structure Evaluations

According to the original construction drawings, the block walls are 8-in. thick and are reinforced horizontally with "K-web" joint reinforcing at every third course. No vertical wall reinforcing was specified in the plans except for four #5 vertical bars with #3 ties at each block pilaster. The T & G diaphragm stresses will be low since the beams provide a continuous tie across the building in the transverse direction. The upper walls of this building are tied together with a continuous bond beam through the block pilasters which contain two #5 bars.

The block wall-to-roof diaphragm connection along the longitudinal walls consists of T & G attachments to a top plate bolted to the top of the block wall. This connection appears to be adequate, since the roof beams provide a continuous tie across the diaphragm, thus lowering the diaphragm stresses. A detail showing the roof connection to the northwest and southeast elevations was not found. The horizontal diaphragm chord steel is several feet below the actual roof level. This connection should be investigated further to determine if the upper portion of these walls is properly tied to the roof. At this time we do not believe that infilling the c.m.u. and providing additional vertical reinforcement would be warranted.

The shear capacity of the masonry walls does not appear to be critical, except possibly at the northeast elevation due to the long window penetrations. This should be investigated further to determine if sufficient shear capacity exists. Based upon these preliminary findings, it may be necessary to reduce the size of the window penetrations in the northeast elevation, and this work has been included in the cost projections. The foundation of the structure is adequately reinforced.

Using the ATC-14 reinforced masonry checklist, this structure has some weak links due mainly to lack of vertical reinforcing. It is recommended that the structure be analyzed further to assess its potential seismic risk. Within the limitations of this preliminary investigation, we rate this structure as having a moderate vulnerability to seismic damage in a Class I event.

5.9.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This equipment is located in the center of the building. The chlorinator cabinet is framed out of light weight plastic and is anchored at all four corners. Most of the piping connected to the equipment is plastic, which has some desirable flexibility for movement. The steel piping, however, is not that flexible and could rupture during an earthquake. The reliability of this equipment could be improved by providing flexible couplings on the piping. Nonetheless, we consider this equipment to have a low seismic vulnerability.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks could slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method of preventing this hazard is to strap the chlorine tanks around their center. The strap should have an easy de-coupling connector so that the tanks can be readily moved. An alternative solution is to construct removable reinforced bollards at the ends of the tank rack, and tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

Miscellaneous Equipment

The chlorine panel in the control room is adequately anchored to the glu-lam beams at the top. The tops of the locker room lockers need to be tied to the basement wall.

5.9.4 Summary and Conclusions

a) From this preliminary structural investigation, we have identified several possible weak links that need to be investigated further. After a more in-depth engineering analysis of the roof connections and shear stress capacity of the masonry walls, some remedial strengthening of the building may be required.

b) Most of the equipment within the chlorination building has a low vulnerability to seismic damage. The lockers need to be

tied to the basement wall for stability. The chlorine tanks should be prevented from sliding off their supports.

c) Table 5.9-1 summarizes the results of the seismic evaluations. An estimate of remedial upgrade costs is presented in Table 5.9-2.

Table 5.9-1

Summary of Preliminary Seismic Evaluations
Tolt South Fork Chlorination Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	More analysis
2	Chlorinators	1	1	Consider flexible couplings
3	Chlorine Tanks	2	1	Provide strap
4	Lockers	2	1	Tie to wall

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 5.9-2

Cost Estimate

Location: Tolt South Fork Chlorination Building

Date Constructed: 1961

Priority: High

Seismic Strengthening Objective:

1) Tie roof diaphragm to NW and SE walls
2) Increase shear capacity of northeast masonry wall
3) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing
4) Prevent lockers from overturning

Upgrade Recommendations:

1) Tie block walls to roof using straps
2) Infill window bays in the northeast wall
3) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and tube steel rails paralleling the tank rack on both sides
4) Tie lockers to wall

Additional Investigation Required:

1) Block wall-to-roof diaphragm connection
2) Shear capacity of northeast masonry wall
3) Lack of vertical reinforcing on seismic performance of structure

Assumption: Construction Performed Outside (Except for securing chlorine tanks and lockers)

Cost Estimate:

1)	Additional Investigation	$4,000
2)	Engineering	19,300
3)	Construction Engineering	12,800
4)	Construction	25,000
	Subtotal:	61,100
5)	Sales Tax (8.1%)	2,020
	Total:	$63,120

Accuracy of Estimate: ±35%

5.10 Corrosion Treatment Building

5.10.1 Facility Description

The Tolt Corrosion Treatment Facility was constructed in 1980 and injects chemicals (lime and soda ash) into the pipelines from the Tolt Regulating Basin to make the water non-corrosive. The facility is located northwest of the Tolt Regulating Basin. The Tolt Corrosion Treatment facility also houses the backup generators for this facility and for the chlorine building. The structure measures 22 ft. by 60 ft. and is approximately 14-ft. tall, and has two interior c.m.u. partition walls. The foundation of the building appears to be adequately reinforced. The walls of the structure are 8-in. reinforced fully-grouted c.m.u. The reinforcing provided in the block walls is consistent with the current UBC minimum requirements. The roof of the building is constructed from 2 x 8-in. roof joists at 24-in. o.c. on center with 3/4-in. plywood attached. The lime storage bin extends about 30 ft. above the roof and is about 15 ft. in diameter. The storage bins are supported on four wide flange steel columns anchored to the concrete foundation. The storage bin anchorage appears to have been designed by the manufacturer to resist seismic forces.

5.10.2 Structure Evaluations

According to the original construction drawings, the block walls are fully grouted and adequately reinforced to provide a large in-plane shear capacity during an earthquake. This structure rated adequately when evaluated with the ATC-14 reinforced masonry checklist, with one exception. The penetrations in the roof diaphragm for the storage bins are large and could possibly limit the available perpendicular-to-wall bracing provided by the wood diaphragm. This should be analyzed further to determine if this poses a major seismic risk. This structure was recently constructed and has current seismic design features that will provide satisfactory seismic performance. Thus, this structure is judged to have a low vulnerability to seismic damage.

5.10.3 Equipment and Contents Evaluations

Motor Control Panels

The anchorage of the motor control center labelled "T" and the distribution panel "T" needs to be verified. Control panel "T23" needs to be anchored to the floor or wall.

Electrical Panels

The electrical panels and piping appear to be properly anchored.

Miscellaneous Equipment

The lockers in the supply room should be bolted to the floor or wall.

Fluosilicic Acid Tanks

Adjacent to the Corrosion Treatment Facility is a separate area which contains two fluosilicic acid tanks. Each tank has a capacity of 5,000 gallons and is 10 ft. in diameter with an overall height of 11 ft. 3 in. The tank shell consists of 1/4-in. thick steel plate with a 3/16-in. thick rubber liner. Each tank is anchored to four 1-ft. high concrete pads with 3/4-in. thick base plates and four 1-in. diameter anchor bolts. The concrete pads, in turn, are an integral part of a concrete mat foundation which measures 14 ft. x 27 ft. x 1 ft. thick. These tanks and their anchorages were determined to be properly designed to withstand a Class I earthquake event, using an assumed seismic acceleration of 0.32g.

5.10.4 Summary and Conclusions

a) In this preliminary structural investigation, a possible weak link has been identified pertaining to the large roof diaphragm opening for the storage bin. A more in-depth engineering analysis of this particular area may indicate that some strengthening of the diaphragm is required; however, at this time we believe that there is a greater likelihood that the diaphragm is adequate, and have not included diaphragm strengthening in the cost projections.

b) Most equipment within the treatment building has low vulnerability to seismic damage. The lockers in the supply room should be bolted to the floor or wall. Control panel "T23" should also be anchored.

c) Table 5.10-1 presents a summary of the results of the seismic evaluations. Table 5.10-2 gives a cost estimate for the recommended corrective measures.

Table 5.10-1

Summary of Preliminary Seismic Evaluations
Tolt South Fork Corrosion Treatment Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	More analysis required for roof diaphragm
2	Equipment	2	1	Anchor lockers and Panel T23

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Note: This facility assigned a high priority because its failure may cause damage to adjacent high priority structures and it houses the emergency generator power supply.

Table 5.10-2

Cost Estimate

Location: Tolt South Fork Corrosion Treatment Building

Date Constructed: 1980

Priority: High

Seismic Strengthening Objectives:

1) Secure control panel "T23"
2) Secure lockers in the supply room

Upgrade Recommendations:

1) Bolt panel "T23" to floor or wall with concrete expansion anchors
2) Bolt lockers to floor or wall with concrete expansion anchors

Additional Investigation Required:

1) Conduct study of large penetrations in roof diaphragm

Assumption: Construction Performed In-House

Cost Estimate:

1)	Additional Investigation	$3,500
2)	Engineering	200
3)	Construction Engineering	100
4)	Construction	1,000
	Subtotal:	4,800
5)	Sales Tax (8.1%)	80
	Total:	$4,880

Accuracy of Estimate: $\pm$25%

5.11 Tolt Standpipe

5.11.1 Facility Description

The Tolt standpipe, constructed in 1962, has a capacity of 25,000 gallons, and is used for washing the screens in the screen house. The structure has a diameter of approximately 17 ft. and a height of 15 ft. Adequate design information was not available pertaining to material specifications, plate thicknesses and foundation dimensions and reinforcement. The cylindrical shell thickness is assumed to be 3/16 in. The bottom plate is also assumed to be 3/16-in. thick. The tank is supported on a ring foundation, assumed to be 4-ft. thick and 18 ft. in diameter and 8-in. wide. Anchorage is provided by four 1½-in. diameter bolts.

5.11.2 Analysis

The computer program ANSYS was used to analyze this standpipe. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. In addition, the effects of the soil-structure interaction cannot be ignored because the flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration. The approach and assumptions used in the analysis are described in Section 3.0.

5.11.3 Analysis Results

The fundamental period of vibration of the standpipe is 0.07 seconds and the fluid sloshing period is 7.46 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Table 5.11-1.

Table 5.11-2 shows a comparison of design shear using different criteria.

In our opinion, the Tolt standpipe will remain operable after the Class I earthquake.

5.11.4 Upgrade Recommendations

No upgrade is required for this standpipe. However it is recommended that the following analysis assumptions be verified (estimated SWD engineering labor cost of $300).

Steel plate:	f_y = 27 ksi
	t = 3/16 in.
Concrete:	f'_c = 3,000 psi
Foundation:	4-ft. deep

Table 5.11-1

Tolt Water Storage Standpipe
Analysis Results (Medium Priority)

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.18	Adequate
Anchor Bolts	0.34	Adequate
Soil Compression	0.90	Adequate
Standpipe Status	**Operable**	

Table 5.11-2

Tolt Water Storage Standpipe
Design Base Shear Comparison (Medium Priority)

1988 UBC Code	57.5 kips
Class I Earthquake	77.6 kips
Class II Earthquake	48.1 kips

5.12 Maintenance Building

5.12.1 Facility Description

The Maintenance Building serves as a repair center and a storage area. The metal building, constructed in 1970, is a light-steel moment frame building measuring 50 ft. by 75 ft. The building eave height is approximately 16 ft. The main functions of the building are to provide shelter for equipment and workspace for maintaining that equipment. The building has two, 20-ft. wide and one, 14-ft. wide overhead doors and one entry door at the south elevation. The building has two transverse rigid frames and two framed end walls. The roof diaphragm consists of "Z" purlins spanning between the rigid frames and the framed end walls. This building is comprised of the typical metal building components with the exception of the rigid frame columns which are constructed out of trussed double angles.

5.12.2 Structure Evaluations

Inspection of the metal building revealed that all of bracing and anchor bolts called out in the construction drawings were present and in good condition. In general, this type of light steel frame building has had an excellent performance record in past earthquakes. In our opinion, the building structure has a low vulnerability to seismic damage in a Class I event.

5.12.3 Equipment and Contents Evaluations

The 8-ft. high by 15-ft. long equipment rack in the storage portion of the building should be anchored to the floor slab and adjacent wall. The storage rack at the east wall elevation should be anchored to the wall. The oxygen/acetylene tanks should be tied to the wall.

5.12.4 Summary and Conclusions

a) From this preliminary structural investigation, the Maintenance Building is properly designed to remain functional following a Class I event.

b) The storage racks and oxygen/acetylene tanks should be properly anchored.

c) Table 5.12-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 5.12-2.

Table 5.12-1

Summary of Preliminary Seismic Evaluations
Tolt South Fork Maintenance Building (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Oxygen Tanks	2	2	Anchor to wall
3	Storage Racks	2	2	Anchor

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 5.12-2

Cost Estimate

Location: Tolt South Fork Vehicle Maintenance Building

Date Constructed: 1970

Priority: Low

Seismic Strengthening Objectives:

1) Secure storage racks to floor and wall
2) Anchor oxygen/acetylene tanks to wall

Upgrade Recommendations:

1) Bolt storage racks to floor and wall
2) Tie tanks to wall

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	100
3)	Construction	700
	Subtotal:	900
4)	Sales Tax (8.1%)	60
	Total:	$ 960

Accuracy of Estimate: ±35%

5.13 Summary and Conclusions

5.13.1 Geotechnical

There does not appear to be any major geologic hazards that would threaten the earthquake functionality of the Tolt South Fork facilities. This conclusion is based upon a limited site review of the existing facilities of the complex. Observations and conclusions were not developed specifically regarding the stability of the slopes adjacent to the Tolt Reservoir.

Signs of prior slope instability were observed adjacent to the chlorination building. Minor slope movements in this area appear to be related to subsurface groundwater flows. Potential movements of the slope in the future could possibly damage underground water lines that run between a tank on the hillside and the chlorination building. Subsurface drainage could improve the local stability of the hillside in this area and reduce the potential for damage to these underground water lines. These remedial repairs would have a low priority for repair.

5.13.2 Structures and Equipment

The evaluation of the Tolt South Fork facilities was based on a site inspection, a review of the construction drawings, and an evaluation prepared with the assistance of the ATC-14 method for evaluating the seismic resistance of existing buildings. The objective was to identify weak links that could represent potential seismic hazards. In many cases, the construction documents were incomplete or not detailed enough to draw specific conclusions. In these cases, additional investigations were recommended.

Table 5.13-1 summarizes the facilities studied along with their vulnerability to seismic damage, facility priority, projected cost, and whether the SWD can upgrade in-house or with an outside contractor.

Table 5.13-1

Summary of Upgrade Recommendations
Tolt South Fork Facilities
(Based on 1989 Dollars)

			Estimated Costs								
Facility	Vulnerability[1]	Facility Priority	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Duvall Shop	Low	Low	$ -0-	$ 100	$ -0-	$ 600	$ 700	$ 50	$ 750	±35%	SWD
Upper Control House	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Upper Control Bridge	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Lower Control House	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Turbine Building	Low	Medium	1,600	-0-	-0-	-0-	1,600	-0-	1,600	±15%	
Screen House	Moderate	High	4,000	18,000	12,900	27,000	61,900	2,250	64,150	±35%	Contractor
Chlorination Building	Moderate	High	4,000	19,300	12,800	25,000	61,100	2,020	63,120	±35%	Contractor
Chlorination Bldg. Slope	Low	Medium	4,000	-0-	-0-	3,000	7,000	240	7,240	±35%	SWD
Corrosion Facility	Low	High	3,500	200	100	1,000	4,800	80	4,880	±25%	SWD
Standpipe	Low	Medium	-0-	300	-0-	-0-	300	-0-	300	±25%	SWD
Maintenance Bldg.	Low	Low	-0-	100	100	700	900	60	960	±35%	SWD
Totals:			$17,100	$38,000	$25,900	$57,300	$138,300	$4,700	$143,000		

[1] Vulnerability refers to a combined ranking of both the structure and any associated equipment.

6.0 LANDSBURG DIVERSION

6.1 Facility Description

The Landsburg Diversion serves to divert water from the Cedar River, and is the intermediate facility between the Cedar River watershed and Lake Youngs. The facility is an integral and vital part of the Cedar River supply system, which provides a major portion of the urban area water supply. Chlorination and fluoridation of the water supply occurs here. Disruption of the functions of these facilities would seriously impact the SWD system's capability to deliver potable water to a large urban area.

6.2 Geologic Hazard Assessment

The facilities at the Landsburg diversion dam are located in a wide, U-shaped valley. The site reconnaissance of the facility and observations of nearby road cuts indicate that the near-surface materials consist of sands and gravels of either alluvial or glacial origin (Zones III and II, respectively, on Figure 4.3-1). The majority of the facilities at the Landsburg site are located upon either level ground or ground with moderate relief. The combination of the slight relief to the topography and the generally competent subsurface conditions would preclude concerns for major, earthquake-induced landslides or liquefaction. This conclusion upon the earthquake stability of the soils at the Landsburg site is based upon the visual site reconnaissance, as borings were not available for this location.

The observed performance of the foundations of the various facilities at Landsburg appears to be favorable and substantiates the conclusions on the competency of the subsurface soils. The site reconnaissance of the facilities indicated that the foundations of the structures appear to be performing quite satisfactorily with no major signs of distress. There were, however, hairline cracks (approximately 1/16 in.) observed in most of the structures including the screen house, chlorination building, fluosilicic acid facility, and the diversion dam. These hairline cracks may have been formed as a result of cracking along cold joints or as a result of cracking due to thermal effects. It would generally appear that these cracks are unrelated to the performance of the foundation soils.

6.3 Screenhouse

6.3.1 Facility Description

The Landsburg Diversion Screenhouse is located near the chlorination facility and was constructed in 1930. It serves to screen out debris from the Cedar River. The structure measures 22 ft. by 66 ft. and is approximately 30 ft. above the adjacent ground level. The upper portion of the screenhouse is founded on reinforced concrete extending approximately 28 ft. below the ground surface.

6.3.2 Structure Evaluations

The roof of the Screenhouse consists of a 4-in. reinforced concrete slab spanning about 9.5 ft. between steel "I" beams. The roof slab is not provided with dowels into the supporting concrete shear walls. The window and door penetrations in all of the screenhouse walls are extensive, leaving the structure with only wall piers to resist lateral seismic forces. According to the original construction drawings, the concrete wall piers vary in thickness from 10 to 14 in. These piers are typically vertically-reinforced columns without ties.

When this structure was evaluated using the ATC-14 concrete shear wall checklist, several weak links were discovered that require further evaluation. The amount of vertical and horizontal wall reinforcing does not comply with the minimum standards of the checklist. The roof-to-wall connection is dependent on shear friction to transfer diaphragm shear stresses to the shear walls. The wall height-to-depth ratios are very high, possibly contributing to an overturning problem. An analysis of the shear stress in the transverse end walls for a Class II earthquake indicates that the walls are not overstressed.

Although this structure is rated as having a moderate vulnerability, it is recommended that a more detailed engineering analysis be performed to predict the seismic risk associated with this structure. At this time, we anticipate that additional shear capacity will need to be added to the walls. This could be accomplished by infilling about two window bays per elevation with concrete, with dowels extending into the existing walls.

6.3.3 Equipment and Contents Evaluations

Screening Equipment

The large screening machines are properly anchored to the floor system.

Electrical Panels

The electrical panels and piping appeared to be adequately anchored.

6.3.4 Summary and Conclusions

a) From this preliminary structural investigation, some possible weak links have been identified within the structure of the Screenhouse. A more in-depth engineering analysis of these particular items should be performed. Based upon this preliminary investigation, the Screenhouse is judged to have a moderate vulnerability to seismic damage.

b) The equipment within the Screenhouse building is well-anchored and has a low vulnerability to seismic damage.

c) Table 6.3-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 6.3-2.

Table 6.3-1

Summary of Preliminary Seismic Evaluations
Landsburg Diversion Screenhouse (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	More analysis required
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 6.3-2

Cost Estimate

Location: Landsburg Diversion Screenhouse

Date Constructed: 1930

Priority: High

Seismic Strengthening Objectives:

1) Strengthen lateral shear capacity of walls

Upgrade Recommendations:

1) Infill selected window bays in each elevation

Additional Investigation Required:

1) Detailed seismic analysis of structure to quantify seismic risk

Assumption: Construction Performed Outside

Cost Estimate:

1)	Additional Investigation	$ 4,000
2)	Engineering	19,700
3)	Construction Engineering	12,600
4)	Construction	42,000
	Subtotal:	79,300
5)	Sales Tax (8.1%)	3,400
	Total:	$82,700

Accuracy of Estimate: ±25%

6.4 Chlorination Building

6.4.1 Facility Description

The Landsburg Diversion Chlorination building was originally constructed in 1965 and treats the water from the Cedar River. The original building measured 24 ft. by 31 ft. An addition was completed in 1969 which added an area 16 ft. long by 24 ft. wide. The roof of the building is 3 x 6-in. T & G decking spanning between 4 x 14-in. wood rafter beams supported on c.m.u. pilasters at each wall and a steel column at the center. The rafter beams are anchored to the pilasters with steel plates. The walls of the structure consist of 8-in. c.m.u. reinforced with "K-web" every third course. Vertical reinforcing was not specified in the drawings; however, horizontal bond beams are supplied over the door and window headers. Typical reinforcement for the pilasters are four #5 vertical and #2 horizontal ties at 12-in. o.c. Several of the exterior c.m.u. walls are faced with stone.

Another addition was added in 1988 that provides shelter for the chlorine tanks. This new addition is a light-steel moment frame building, measuring approximately 24 ft. by 60 ft. and extending north from the interface between the chlorine room and the scale room.

6.4.2 Structural Evaluations

Details showing the roof diaphragm connection to the block walls were not provided in the original construction drawings or the drawings developed for the addition. These connection details are important, especially for the exterior walls with the stone facing. Without adequate anchorage to the roof diaphragm, the heavy block walls could lose their out-of-plane support. These connections should be investigated to verify proper detailing. Also, the drawings for the addition completed in 1969 show that the interior masonry walls are not positively anchored at the top, thus creating a potential seismic hazard. Due to the fact that this is a relatively small, single-story structure with interior c.m.u. walls and pilasters, we do not believe that major damage will occur in a Class I seismic event. The light steel moment frame addition appears to be well-built. We expect it to perform well in a Class I event.

According to the ATC-14 reinforced masonry checklist, this structure has some weak links, mainly due to lack of vertical reinforcing and a positive connection from the masonry walls to the roof diaphragm. We recommend that the structure be investigated further to assess its potential seismic risk. We rate this structure as having a moderate vulnerability to seismic damage in a Class I event, and have included work for improving the roof-to-wall connection in the cost projections.

6.4.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This equipment is located near the center of the building. The chlorinator cabinet is framed out of light-weight plastic and is anchored on all four corners. Most of the piping connected to the equipment is plastic, which has some flexibility for movement. The steel piping, however, is not that flexible and could rupture during an earthquake. The reliability of this equipment could be improved by providing flexible couplings on the piping. This equipment has low vulnerability to seismic damage.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feed lines to rupture. One method of solving this potential hazard is to strap the chlorine tanks around their center. The strap should have an easy de-coupling connector to facilitate moving the tanks. An alternative solution is to construct removable reinforced bollards at the ends of the tank rack, and tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

6.4.4 Summary and Conclusions

a) From this preliminary structural investigation, the Chlorination Building has a moderate vulnerability to seismic damage. It is recommended that a more in-depth inspection of the roof-to-wall connection be performed.

b) Most equipment within the chlorination building has a low vulnerability to seismic damage. The chlorine tanks must be secured to prevent them from sliding off their supports.

c) Table 6.4-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 6.4-2.

Table 6.4-1

Summary of Preliminary Seismic Evaluations
Landsburg Diversion Chlorination Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	More analysis required
2	Chlorinators	1	1	Consider flexible couplings
3	Chlorine tanks	2	1	Anchor strap required

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 6.4-2

Cost Estimate

Location: Landsburg Diversion Chlorination Building

Date Constructed: 1963 (Original) and 1969 (Addition)

Priority: High

Seismic Strengthening Objectives:

1) Tie roof to three exterior walls
2) Secure chlorine tanks to prevent feeder lines from rupturing

Upgrade Recommendations:

1) Tie roof to three walls with straps 4-ft. on center
2) Secure chlorine tanks with strap and ratchet arrangement, or construct removable reinforced bollards at the end of the tank rack and install tube steel rails paralleling the tank rack on both sides

Additional Investigation Required:

1) Verify block wall-to-roof diaphragm connection
2) Evaluate lack of vertical reinforcing on seismic performance of structure

Assumption: Construction Performed Outside (Except for securing chlorine tanks)

Cost Estimate:

1)	Additional Investigation	$ 4,000
2)	Engineering	21,300
3)	Construction Engineering	16,600
4)	Construction	40,000
	Subtotal:	81,900
5)	Sales Tax (8.1%)	3,250
	Total:	$85,150

Accuracy of Estimate: ±30%

6.5 Fluosilicic Acid Facility

6.5.1 Facility Description

The fluosilicic acid equipment structure is located adjacent to the Landsburg water fluoridation tanks and is also known as the compressor building. The building is a single-story pumice block-and-mortar structure measuring 7 ft. by 9 ft. with a single, 3-ft. wide door opening. The construction drawings were not available; however, the construction of this building appears very similar to the chlorine and sewage treatment building at Cedar Falls. Typically, the roofs of structures similar to this are constructed of metal decking with about 3-1/2 in. of concrete covering, with the metal decking positively attached to the top of the pumice wall.

6.5.2 Structure Evaluations

Typical construction of these types of buildings consists of 8-in. pumice block without any reinforcing except for around the door lintel. The metal decking is probably attached to the top course of block with anchor bolts. There exist a number of weak points in the structure mainly due to the lack of reinforcing. However, since this building is relatively small and light-weight, it will not experience a large amount of acceleration in an earthquake. Thus, major damage is not expected to result, even though the structure is essentially unreinforced.

With these considerations, we rate this structure as having a low vulnerability to a Class I-type earthquake.

6.5.3 Equipment and Contents Evaluations

The equipment in this facility consists of a compressed air dryer, compressor, and a few power panels. All equipment contained within the compressor building is well anchored and has low vulnerability to seismic damage.

The three fluosilicic acid tanks at Landsburg are enclosed in their own containment area. Each tank has a capacity of 8,000 gallons, and is 10 ft. in diameter with an overall height of 16 ft. 3 in. The tank shell consists of a 1/4-in. thick steel plate with a 3/16-in. rubber liner. Similar to the Tolt fluosilicic acid tank design, two of the tanks are each anchored to four 1-ft. high concrete pads with 3/4-in. thick plates and four 1-in. diameter anchor bolts. The concrete pads are an integral part of the concrete foundation mat, which is triangular in shape, with overall dimensions of 25 ft. 3 in. x 29 ft. 1 7/8 in. x 1 ft. thick. The remaining tank is anchored directly to the concrete foundation mat. These tanks and their anchorages were determined to be properly designed to withstand a Class I earthquake event, using an assumed seismic acceleration of 0.32g.

6.5.4 Summary and Conclusions

a) Without detailed construction drawings, it is difficult to precisely determine the adequacy of this building in resisting earthquakes. Nonetheless, major damage is not predicted and therefore no remedial strengthening measures are required.

b) The two fluosilicic acid tanks and associated equipment is well-anchored and has low vulnerability to seismic damage.

c) Table 6.5-1 summarizes the results of the seismic evaluations.

Table 6.5-1

Summary of Preliminary Seismic Evaluations
Landsburg Fluosilicic Acid Facility (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure (Low Priority)	1	1	O.K.
2	Fluosilicic acid tanks (High Priority)	1	1	Well anchored
3	Equipment	1	1	Well anchored
4	Electric control panel	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

6.6 Generator Building

6.6.1 Facility Description

The generator building at Landsburg, constructed in 1979, is of light-steel, moment-frame construction measuring 11 ft. wide by 15 ft. long. The building eave height is approximately 10 ft. The main function of this building is to provide shelter for the auxiliary generator power supply servicing Landsburg. The emergency generator is a diesel engine with its fuel supply located directly outside of the metal building. The power supply lines are underground.

6.6.2 Structure Evaluations

The original construction documents for the metal building were unavailable, however, from the field inspection the structure appears to conform to normal industry practices for its size. In general, this type of light steel frame building has had an excellent performance record in past earthquakes. Thus, we rate the building structure as having a low vulnerability to seismic damage in a Class I event.

6.6.3 Equipment and Contents Evaluations

Diesel Generator

The diesel generator has low vulnerability to seismic damage since it is well-anchored to the floor slab on grade.

Electrical Control Panels

The two electrical control power panels along the longitudinal wall of the building are adequately anchored to the floor slab.

Diesel Supply Tank

The fuel tank supplying the generator is located outside of the building. This tank is very susceptible to seismic damage since it is above ground and the frame supporting the tank is light. In a Class I earthquake, this tank may overturn and cause the supply lines to rupture, thus rendering the emergency power supply inoperable. We recommend that a new tank support be designed and installed as soon as possible.

6.6.4 Summary and Conclusions

a) The Generator Building is properly designed to withstand a Class I event.

b) The supports for the fuel tank supplying the generator should be upgraded to withstand a Class I seismic event.

c) A wood cabinet within the generator building is not anchored and presents a seismic hazard to the equipment. This cabinet should be tied back to the wall.

d) Table 6.6-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 6.6-2.

Table 6.6-1

Summary of Preliminary Seismic Evaluations
Landsburg Generator Building and Equipment (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Diesel fuel tank	3	2	Upgrade anchorage
3	Electric control panel	1	1	Adequate anchorage
4	Wood Cabinet	2	1	Tie to wall

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 6.6-2

Cost Estimate

Location: Landsburg Generator Building

Date Constructed: 1979

Priority: High

Seismic Strengthening Objectives:

1) Prevent fuel storage from overturning and rupturing fuel supply lines.
2) Anchor wood cabinet to wall

Upgrade Recommendations:

1) Provide a new support configuration to secure fuel tank
2) Bolt wood cabinet to wall framing

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 500
2)	Construction Engineering	100
3)	Construction	1,600
	Subtotal:	2,200
4)	Sales Tax (8.1%)	130
	Total:	$2,330

Accuracy of Estimate: ±35%

6.7 Landsburg Elevated Tank

6.7.1 Facility Description

The Landsburg elevated water storage tank, built in 1930, has a capacity of 48,500 gallons and is used for washing the screens in the screenhouse and for the chlorine injector water. The structure, shown in Figures 6.7-1 and 6.7-2, has a diameter of approximately 22 ft., and the center of gravity for the structure and contents is located at 111 ft. above grade level. The elevated tank shell is supported by a ring girder resting on four braced columns with a cross sectional area of approximately 18 sq. in. The rod braces have a cross section of 1 sq. in. There is a 43-in. diameter central riser with an assumed cross section of approximately 48 sq. in. The columns are supported by concrete spread footings.

6.7.2 Analysis

The three dimensional model developed for use with the computer program ANSYS is shown in Figure 6.7-3. The tank shell connection at the top of the columns was modeled by coupling the equivalent ring beam to the top of the columns, and using constraint equations to couple the horizontal displacements without further restraining the rotations. This assumption is justified because at those connections, forces are transferred primarily by shear along the tangential surface of the tank, where the tank shell is very rigid. The rod braces were assumed to take tension only.

The connections at the base of each column were modeled as hinges. The riser has no anchor bolts, and rotational springs were included in the model at the base of the riser.

Potential sloshing of the water contents was modeled using Housner's method. The weight of the water was distributed to the columns and riser in accordance to their tributary area.

6.7.3 Analysis Results

The fundamental period of vibration of the tank is 1.38 seconds and the fluid sloshing period is 2.82 seconds. This places the tank period in the descending branch of the response spectra (Figure 4.3-3). Any softening of the tank (i.e. yielding) will contribute to lowering the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 6.7-1 and 6.7-2. Table 6.7-3 shows a comparison of design base shears using different criteria.

In our opinion, the structure will not remain operable after the Class I earthquake. Yielding of the braces poses a moderate

problem, however, uplift of the column footings will result in a loss of lateral load capacity.

The structure will remain operable after the Class II earthquake. Moderate soil overstress will take place resulting in lower effective loads in the structure.

6.7.4 Upgrade Recommendation

Increase the hold-down capacity of the footings by placing new concrete over the existing footings as shown in Figure 6.7-6. This new configuration will permit the diagonal braces to yield during a Class I seismic event; however yielding of the rod braces will not compromise the safety of this structure. The expected upgrade cost of this remedial upgrade solution is presented in Table 6.7-4.

Table 6.7-1

Landsburg Elevated Water Storage Tank
Analysis Results (Medium Priority)

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces (tension)	1.64	Yielding
Riser (tension in bending)	N.A.	Riser not effective in tension
Riser (compression)	0.17	Adequate
Columns (compression)	0.92	Adequate
Soil Under Columns (bearing)	2.06 (allowable)	Adequate, movement expected at the foundation level
Column Footing (uplift)	5.3 kips	Column footing uplifts
Soil Under Riser (bearing)	1.12 (allowable)	Minor soil overstress
Elevated Tank Status	**Not Operable**	

Table 6.7-2

Landsburg Elevated Water Storage Tank (Medium Priority)
Analysis Results

Class II Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces (tension)	1.00	Adequate
Riser (tension in bending)	N.A.	Riser not effective in tension
Riser (compression)	<0.17	Adequate
Columns (compression)	<1.00	Adequate
Soil Under Columns (bearing)	1.31 (allowable)	Adequate, minor soil overstress
Soil Under Riser (bearing)	<1.00 (allowable)	Adequate
Elevated Tank Status	**Operable**	

Table 6.7-3

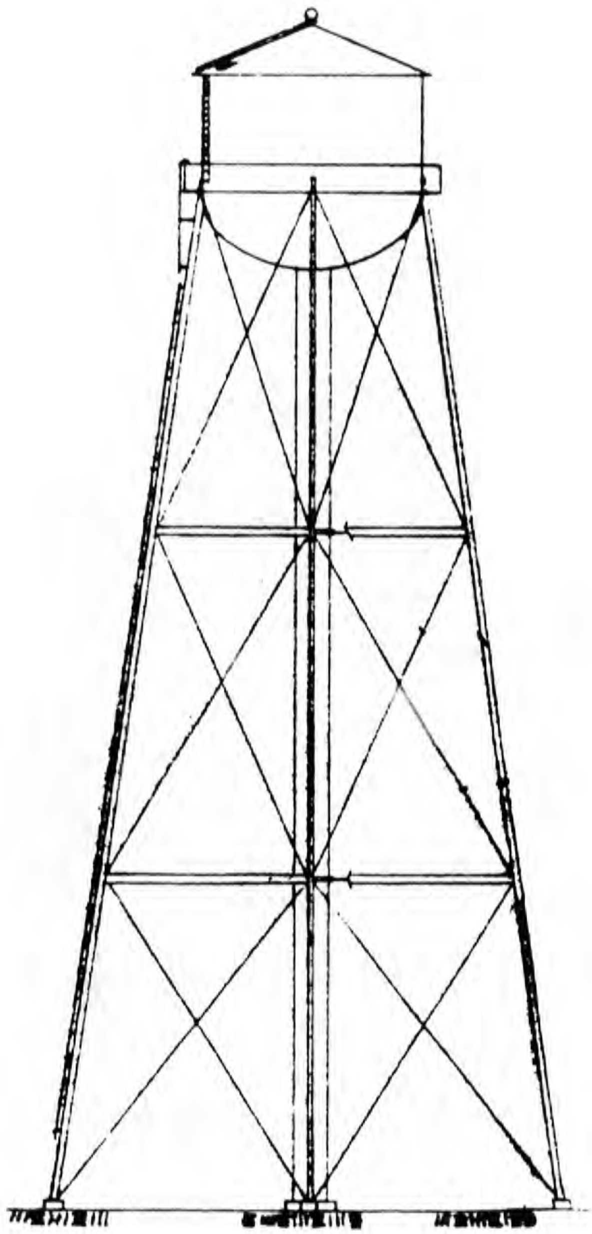
Landsburg Elevated Water Storage Tank (Medium Priority)
Design Base Shear Comparison

1988 UBC Code	75.2 kips
Class I Earthquake	66.8 kips
Class II Earthquake	51.4 kips

Table 6.7-4

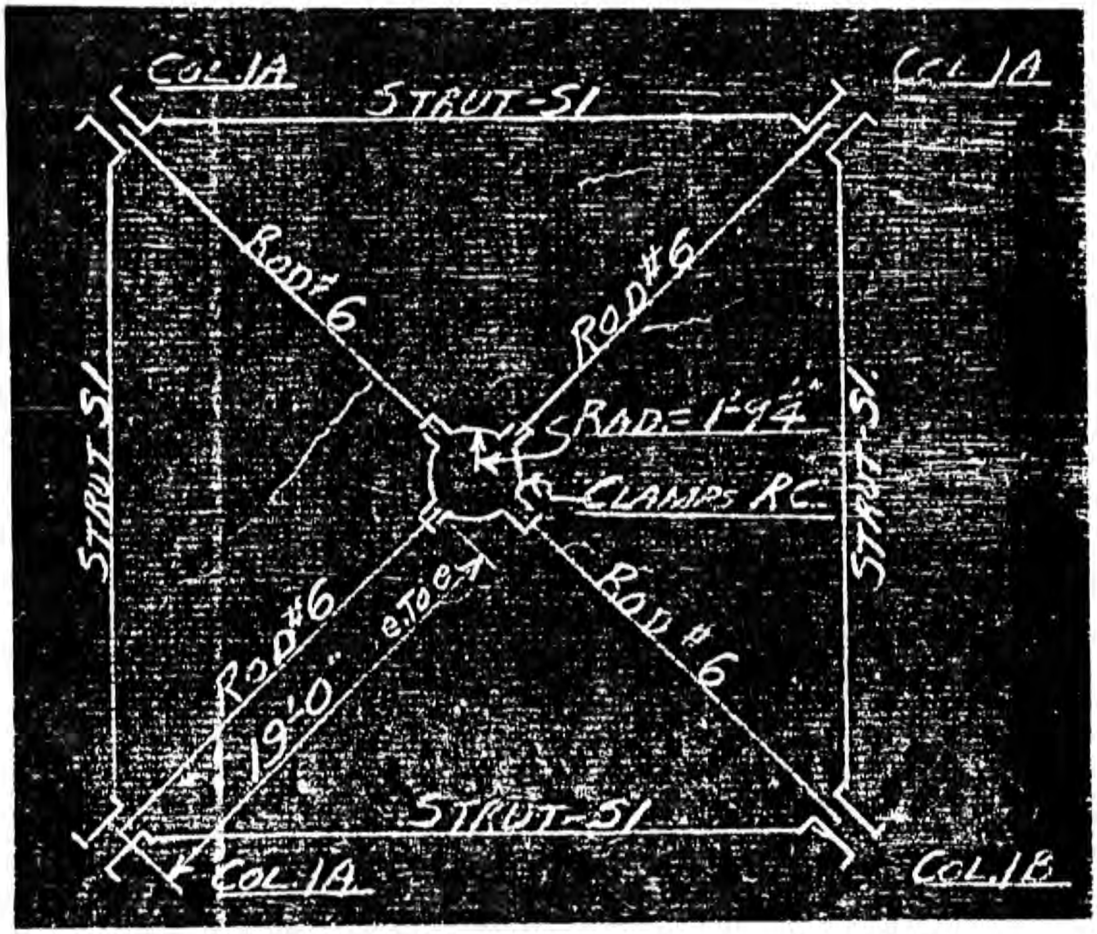
Landsburg Elevated Water Storage Tank (Medium Priority)
Cost Estimate

Engineering	$13,200
Construction Engineering	8,500
Construction	25,000
Subtotal	$ 46,700
Sales Tax (8.1%)	2,020
Total	$48,720
Accuracy of Estimate	±25%



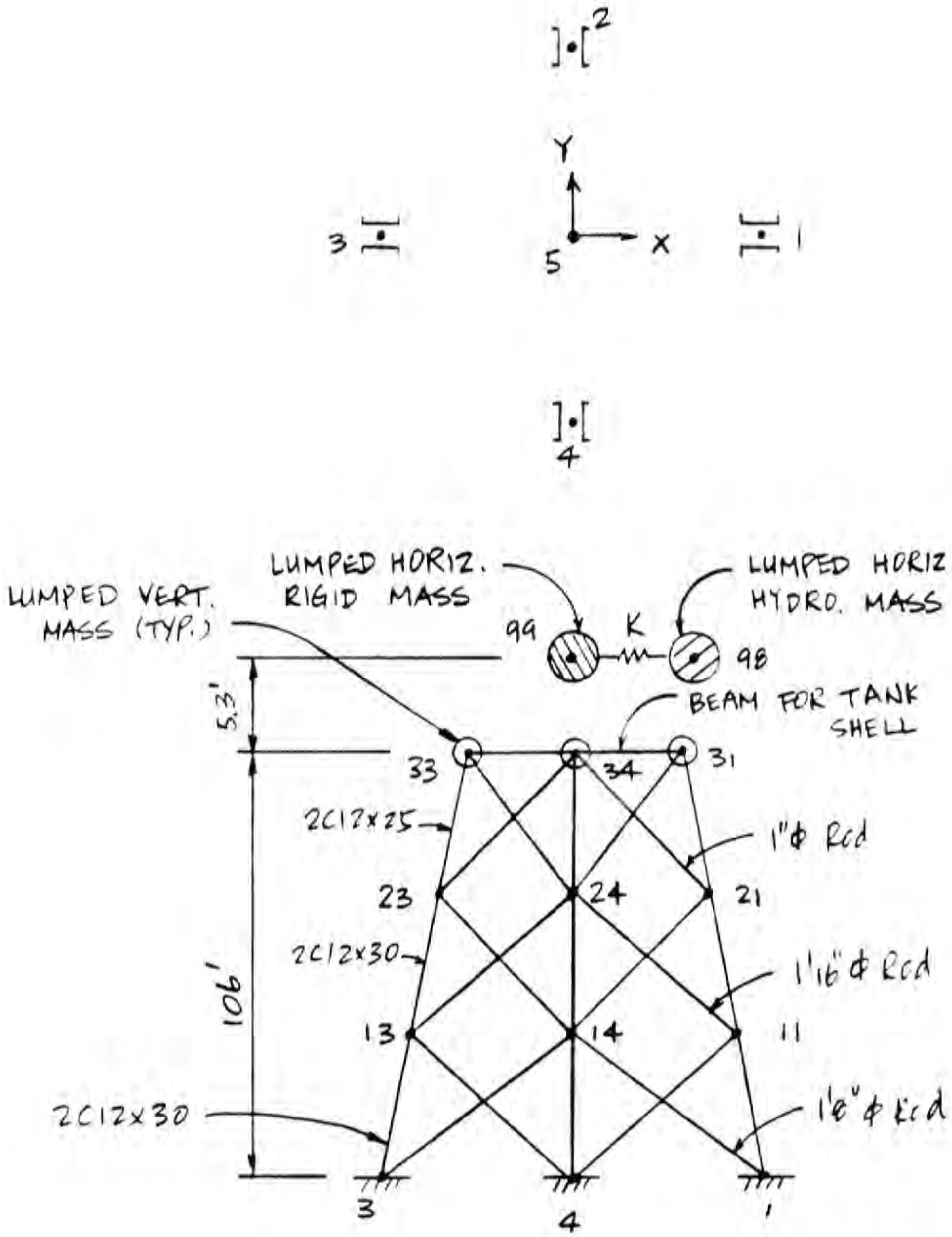
Landsburg Elevated Water Storage Tank
Elevation View

Figure 6.7-1



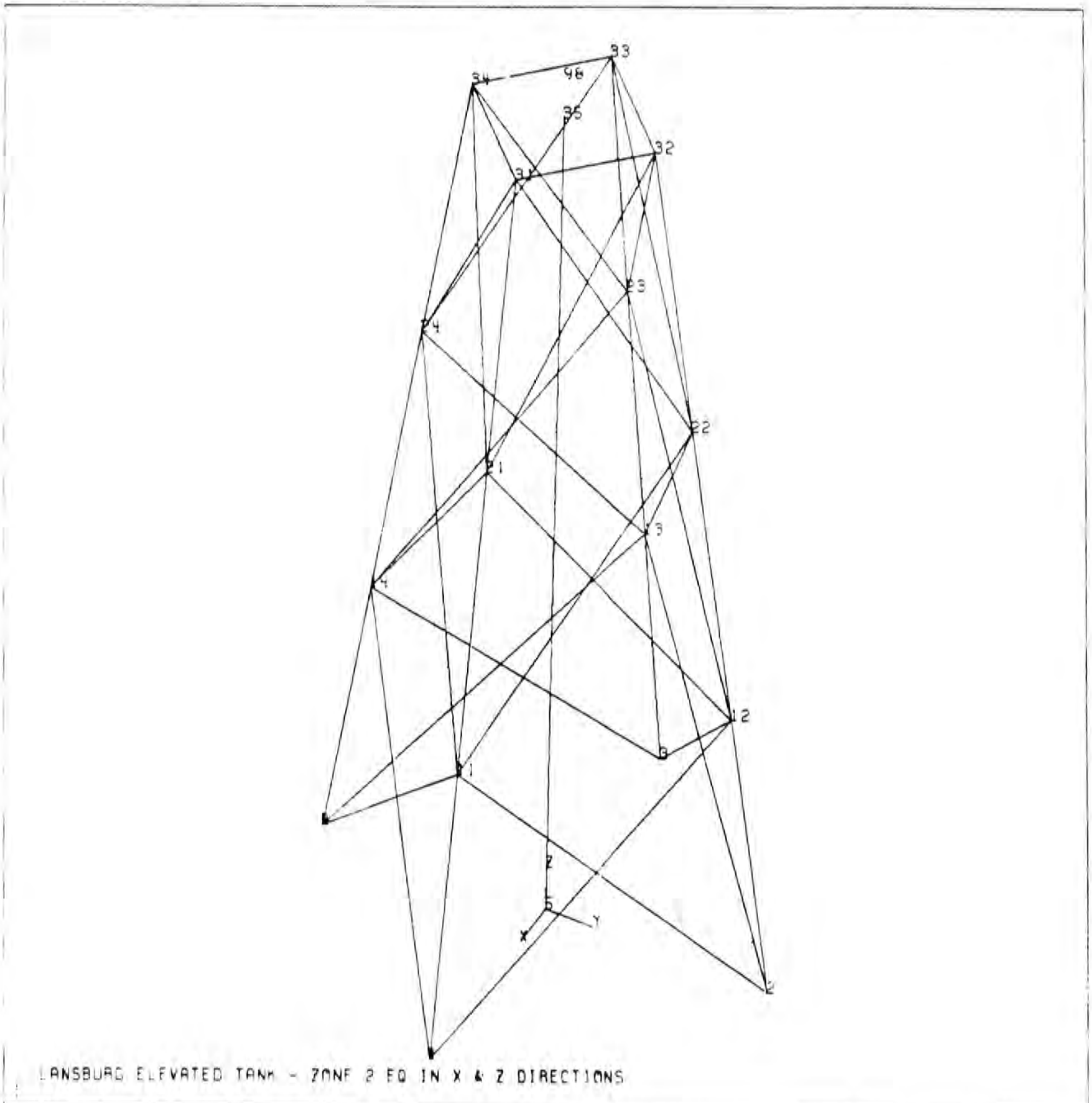
Landsburg Elevated Water Storage Tank
Plan View

Figure 6.7-2



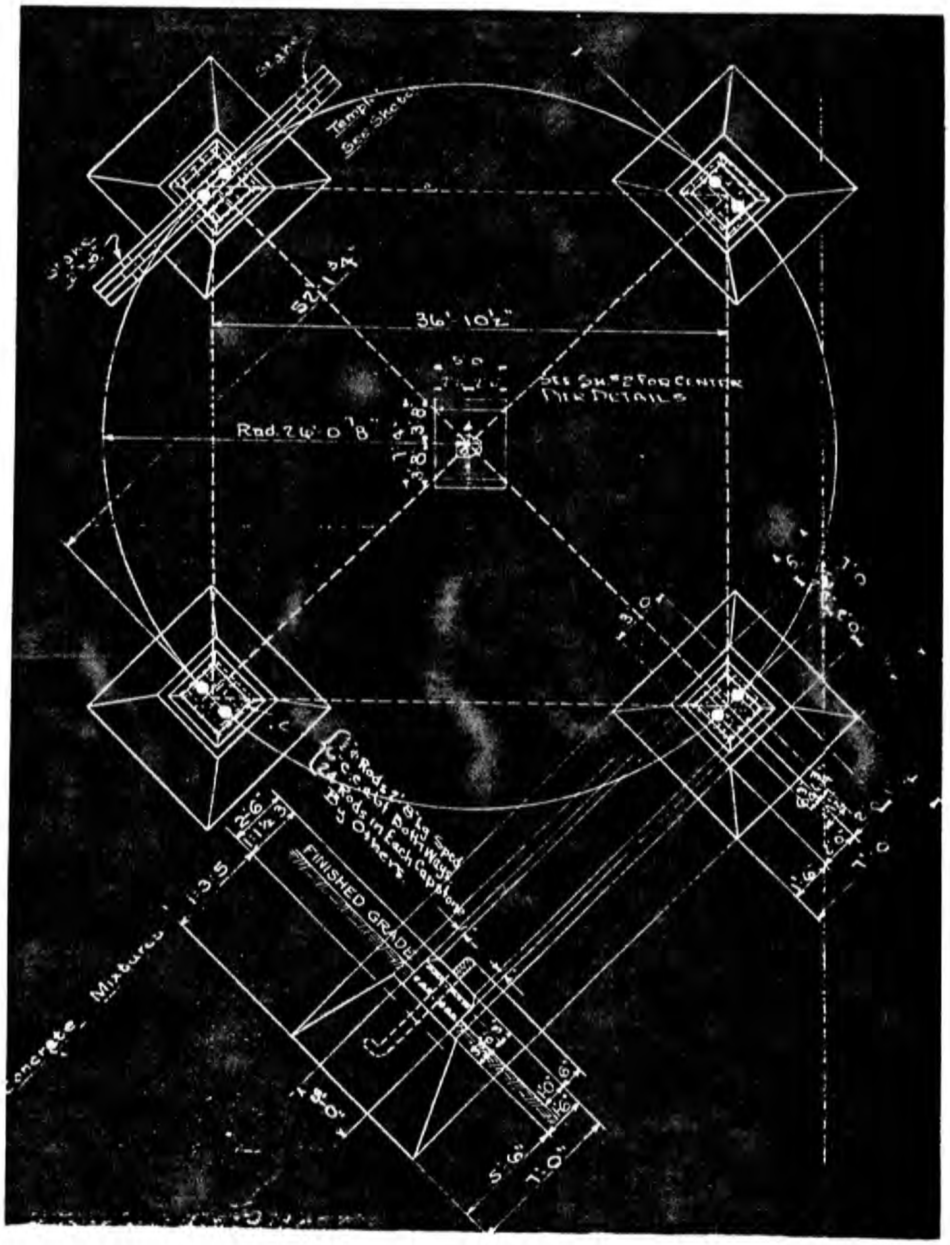
Landsburg Elevated Water Storage Tank
Mathematical Model

Figure 6.7-3



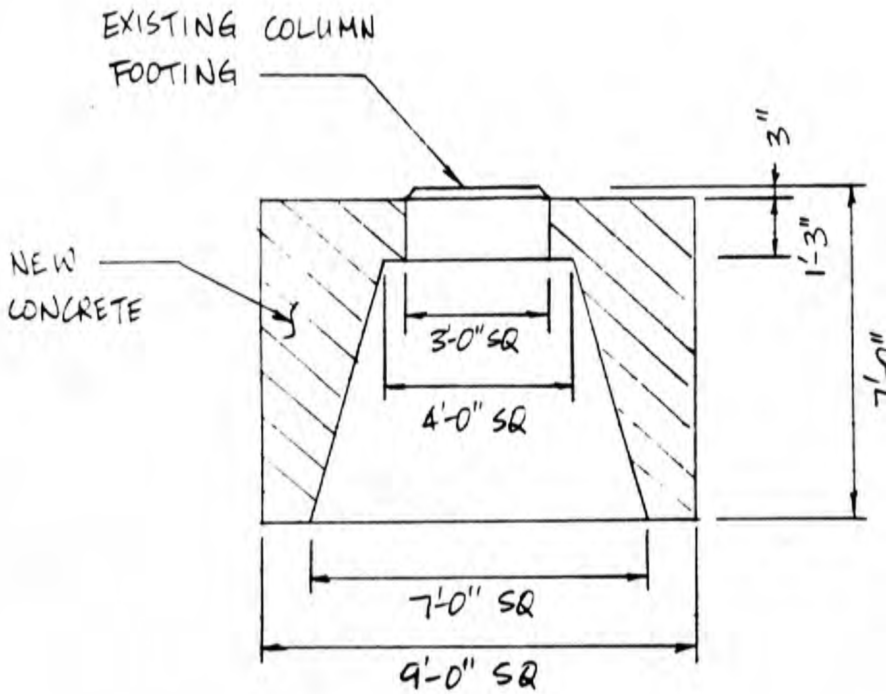
Landsburg Elevated Water Storage Tank
Computer Plot

Figure 6.7-4



Landsburg Elevated Water Storage Tank Base

Figure 6.7-5



Landsburg Elevated Water Storage Tank
Foundation Upgrade Alternative

Figure 6.7-6

6.8 Tunnel Gate House

6.8.1 Facility Description

The Landsburg tunnel gate house facility is located between the Cedar River Watershed and Lake Youngs. The original portion of the gate house was constructed in 1930 and an addition measuring 19-ft. x 21-ft. was constructed in 1975. Within the gate house, a 96-in. diameter steel water pipe splits into two separate 78-in. diameter pipes. The original structure has cast-in-place reinforced concrete walls on continuous footings with a basement about 12 ft. below grade. The roof slab is approximately 11 ft. above an intermediate steel floor framed on steel beams. The plans available for this building were difficult to read, but it appears that the original building measured 12 ft. by 21 ft. in plan. The concrete slab roof in the original portion is supported by two center steel beams. The original window openings have been filled in with c.m.u. block.

The addition consists of 12-in. thick reinforced concrete retaining walls below grade with 8-in. thick reinforced concrete walls above grade. The roof consists of a 3-1/2 in. concrete slab over corrugated metal decking supported on steel roof beams.

6.8.2 Structure Evaluations

From the construction documents for the original building, the 12 in. exterior concrete perimeter basement walls appear to be reinforced with #5 bars at 6-in. o.c. both ways on the inside face, which is adequate. The roof slab is keyed into the perimeter walls, which will provide adequate strength to transfer the diaphragm stresses into the walls. The presence of reinforcing in the upper 8 in. concrete walls was not confirmed from a review of construction drawings; this should be investigated further to insure that adequate reinforcing is provided. The drawings were not complete enough to make a determination as to the performance of this structure in a Class I seismic event; however, in general these types of one-story concrete buildings are relatively rugged and perform well in earthquakes. Thus, this structure has a moderate vulnerability to seismic damage in a Class I earthquake, based upon the available information. Some cracking of the concrete walls may occur; however, the structural integrity of the building should be sustained.

The new wall construction of the addition was formed and placed against the existing structure. The walls were doweled to the existing structure and the addition should therefore perform well in a Class I seismic event.

6.8.3 Equipment and Contents Evaluations

Electric Control Panels

There are several electric panels located within both the older and newer portions of the building. Most of the electrical equipment is enclosed in rigid sheet metal cabinets attached to the concrete walls. This equipment has a low vulnerability to damage.

6.8.4 Summary and Conclusions

a) From the preliminary investigation of the original construction documents, it was not possible to obtain adequate information on some important details. However, when the ATC checklist for concrete structures was evaluated, this building rated as marginally adequate. Further investigation of this structure is warranted; however, it is not anticipated that structural strengthening would be required.

b) The electrical equipment is well-supported and has low vulnerability to damage.

c) Table 6.8-1 summarizes the results of the seismic evaluations. There are no anticipated upgrade costs required for this facility.

Table 6.8-1

Summary of Preliminary Seismic Evaluations
Landsburg Tunnel Gate House (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Potential wall cracks
2	Electric Control Panels	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 6.8-2

Cost Estimate

Location: Landsburg Tunnel Gate House

Date Constructed: 1930 (Original) and 1975 (Addition)

Priority: High

Additional Investigation Required:

1) Evaluate potential lack of reinforcing in upper walls on seismic performance of structure

Cost Estimate:

1)	Engineering	$3,600
	Total:	$3,600

Accuracy of Estimate: ±20%

6.9 Summary and Conclusions

6.9.1 Geotechnical

There does not appear to be any major geologic hazards that would threaten the earthquake functionality of the facilities at Landsburg. This conclusion is based upon a limited site review of the existing facilities. Observations and conclusions were not developed specifically regarding the stability of the slopes along the reservoir associated with the Landsburg facilities.

6.9.2 Structures and Equipment

The evaluation of the Landsburg Diversion facilities was based on a site inspection, a review of the construction drawings, and an evaluation performed with the assistance of the ATC-14 method for evaluating the seismic resistance of existing buildings. The objective for employing this methodology was to identify any potential weak links that could represent seismic hazards. In many cases, the construction documents were incomplete or not detailed enough to draw precise conclusions. In these cases, additional investigations were recommended.

Table 6.9-1 presents the facilities studied, with their vulnerability to seismic damage, facility priority, upgrade cost, and whether the remedial work can be performed by the SWD or an outside contractor.

Table 6.9-1

Summary of Upgrade Recommendations
Landsburg Diversion Facilities
(Based on 1989 Dollars)

			Estimated Costs								
Facility	Vulnerability[1]	Facility Priority	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Screen House	Moderate	High	$ 4,000	$ 19,700	$12,600	$ 42,000	$ 79,300	$ 3,400	$ 82,700	±25%	Contractor
Chlorination Building	Moderate	High	4,000	21,300	16,600	40,000	81,900	3,250	85,150	±30%	Contractor
Fluosilicic Acid Fac.	Low	Low	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Generator Building	Low	High	-0-	500	100	1,600	2,200	130	2,330	±35%	SWD
Elevated Tank	High	Medium	-0-	13,200	8,500	25,000	46,700	2,020	48,720	±25%	Contractor
Tunnel Gate House	Moderate	High	3,600	-0-	-0-	-0-	3,600	-0-	3,600	±20%	
Totals:			$11,600	$54,700	$38,800	$108,600	$213,700	$8,800	$222,500		

(1) Vulnerability refers to a combined ranking of both the structure and any associated equipment.

7.0 CEDAR FALLS

7.1 Facility Description

The Cedar Falls facility, located about 5 miles southeast of North Bend, supports the staff managing the Cedar River Watershed and Chester Morse Lake. The Office Shop, the Sewage Treatment and Chlorine Building, and the Well Supply and Pump Building comprise the major site facilities. Disruption of the function of these facilities, while hampering staff work, would not of itself disrupt water supply capabilities.

7.2 Geologic Hazard Assessment

The facilities at Cedar Falls are located in a U-shaped valley underlain by sand and gravel deposits of glacial origin (Zone II on Figure 4.3-1). Based upon probing during the visual site reconnaissance, it was concluded that these underlying sands and gravels were very dense. The combination of the slight topographic relief and the generally competent subsurface conditions would generally preclude major concerns for earthquake-induced landslides or liquefaction.

The observed performance of the foundations of the facilities at Cedar Falls appears to be favorable and substantiates the conclusions on the competency of the underlying soil conditions. Specifically, the foundations of the structures appear to be performing satisfactorily, without any signs of major distress.

The access road leading from Rattlesnake Lake to the Masonry Dam was also reviewed during the site geological reconnaissance. The slopes adjacent to this road appear to be relatively stable, showing no major signs of distress. Major landslides that have occurred in this area, specifically the Boxley landslide, appear to be related to the operational level of Chester Morse Lake. These landslides have been discussed in great detail in other reports provided to the SWD.

7.3 Office Shop

7.3.1 Facility Description

The Cedar Falls Office Shop Building is located near Cedar Falls Road S.E. in the Cedar Falls Area. The structure is composed of several modular units and measures approximately 60-ft. wide by 104-ft. long. The original building was constructed in 1954, and an addition measuring 18 ft. by 19 ft. was added in 1976. The building is used as an administration office, maintenance area, and living quarters. Much of the lower portion of the building is constructed from 8-in. block walls. The sleeping dormitory comprises the second story portion of the building, measuring 20-ft. wide by 38-ft. long. The entire roof of the building consists of 3 x 6-in. T & G decking supported on wood beams.

7.3.2 Structure Evaluations

The original construction documents showed that the 8-in. block walls were built with horizontal joint reinforcing at 24-in o.c. The only vertical reinforcing is at the block pilasters. The 7-ft. high block wall at the entry way is not supported at the top and could suffer some damage in an earthquake. Doweling of the masonry walls to the perimeter foundation was not specified; however, this is probably not a major concern since the walls are long compared to their height. The connection details shown in the drawings appear to be adequate to resist seismic forces and to tie the building together. Many of the walls are constructed from wood framing. Typically, 2 x 4-in. framing with 1-in. thick diagonal wood sheathing is applied to the exterior. This type of construction should be adequate to resist the Class I seismic forces.

Using the ATC-14 reinforced masonry building checklist, this structure has several weak links, mainly due to the lack of adequate vertical reinforcing and positive ties from the masonry walls to the roof and floor diaphragms. Most of the major seismic connections appear to be adequately detailed. Thus the building structure is rated as having a moderate vulnerability to seismic damage in a Class I event, though structural functionality should be sustained. In our opinion, structural seismic strengthening is not required.

7.3.3 Equipment and Contents Evaluations

All of the tall lockers within the building should be bolted to the walls or the floor to prevent overturning. The parts storage racks should be bolted to the floor. All of the storage parts racks and lockers should be bolted down to the floor slab, and the heavier equipment should be located on the lower shelf.

7.3.4 Summary and Conclusions

a) From this preliminary structural investigation, there are a few minor problems with the office structure. Though these weaknesses are sufficient to rate the structure as moderately vulnerable to seismic damage from a Class I event, we do not believe that the ensuing structural damage would be sufficient to warrant structural seismic strengthening.

b) The lockers and storage racks should be anchored to the walls or floors.

c) Table 7.3-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 7.3-2.

Table 7.3-1

Summary of Preliminary Seismic Evaluations
Cedar Falls Office Shop (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	O.K.
2	Lockers	2	1	Anchor
3	Storage racks	2	1	Anchor

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 7.3-2

Cost Estimate

Location: Cedar Falls Office Shop

Date Constructed: 1954 (Original) and 1976 (Addition)

Priority: Medium

Seismic Strengthening Objectives:

1) Secure lockers and storage racks

Upgrade Recommendations:

1) Bolt lockers and storage racks to walls or floors

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	\$ 300
2)	Construction Engineering	200
3)	Construction	2,500
	Subtotal:	3,000
4)	Sales Tax (8.1%)	200
	Total:	\$3,200

Accuracy of Estimate: ±30%

7.4 Sewage Treatment Building

7.4.1 Facility Description

This facility, constructed in 1967, is located within the Cedar Falls watershed area northeast from the existing SWD Cedar Falls facilities. The building is a single story pumice block and mortar type structure, measuring 6 ft. wide by 6 ft. long. The roof is constructed of metal decking with 3.5 in. of concrete covering and measures about 10 ft. x 10 ft. The metal decking is attached to the top of the pumice wall.

7.4.2 Structure Evaluations

According to the construction drawings, the exterior bearing walls consist of 8-in. pumice block without any reinforcing except around the door lintel. The metal decking is attached to the top course of block with anchor bolts of unknown quantity and size. Doweling of the walls to the foundation is not specified in the plans. Only one door opening exists in the building which is 3-ft. wide. If the ATC-14 rapid evaluation check list were rigorously used, it would point out several weak links in the structure mainly due to the lack of reinforcing. However, since this building is relatively small and light weight, it will not experience a great amount of seismic acceleration and thus major damage.

With these considerations, we rate this structure as having a low to moderate vulnerability to a Class I type earthquake.

7.4.3 Equipment and Contents Evaluations

This building is primarily used for the storage of calcium hypochlorite dry chemical in containers. There are no other significant equipment-related items.

7.4.4 Summary and Conclusions

a) Since this structure is small and light, even though the structure is unreinforced no major seismic damage is anticipated. Remedial strengthening of this building is not warranted.

b) Table 7.4-1 summarizes the results of the seismic evaluations.

Table 7.4-1

Summary of Preliminary Seismic Evaluations
Cedar Falls Sewage Treatment Building (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Unreinforced

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

7.5 Well Supply and Pump Building

7.5.1 Facility Description

This facility is located within the Cedar Falls watershed area northwest of the existing Cedar Falls facilities. At the time of the site inspection, the building was under construction with all four 8 in. c.m.u. walls built and roof joists in place. The building is a single story c.m.u. block and mortar type structure measuring 12 ft. wide by 25 ft. long with an interior 8 in. c.m.u. bearing wall near the center of the building which extends to the roof plywood diaphragm.

7.5.2 Structure Evaluations

According to the construction drawings, the exterior bearing walls consist of 8 in. c.m.u. walls with horizontal bond beams with #5 reinforcing bars at about 24 in. o.c. and "K-web" reinforcing at every second course. The vertical reinforcing is #5 bars at 48 in. o.c., with vertical bars at openings and three bars in the corners of the building as typical. The wall reinforcing in this building as shown on the contract documents, is consistent with current seismic design methods.

The top of the c.m.u. walls have a bond beam with two continuous #5 bars which assist in the lateral stability of the structure. In using the ATC-14 rapid evaluation check list, no weak links were discovered. With these considerations, we rate this structure as having a low vulnerability to a Class I type earthquake.

7.5.3 Equipment and Contents Evaluations

At the time of the inspection, the facility equipment was not in place. However, a review of the contract documents indicated that the generator and pump foundations had adequate anchorage.

7.5.4 Summary and Conclusions

a) From the contract documents, this structure is properly detailed so as to provide adequate resistance to earthquakes. If the equipment is installed and anchored with the same amount of care, little damage should occur.

b) Table 7.5-1 summarizes the results of the seismic evaluations.

Table 7.5-1

Summary of Preliminary Seismic Evaluations
Cedar Falls Well Supply and Pump Facility (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	Well built
2	Pump supports	1	1	Well anchored
3	Generator	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

7.6 Summary and Conclusions

7.6.1 Geotechnical

There does not appear to be any major geologic hazards that would threaten the earthquake functionality of the facilities at Cedar Falls. This conclusion is based upon a limited site review of the existing site facilities. Observations and conclusions were not developed specifically regarding the stability of the slopes adjacent to Chester Morse Lake.

7.6.2 Structures and Equipment

The evaluation of the Cedar Falls facilities was based on a site inspection, a review of the construction drawings, and an evaluation performed with the assistance of the ATC-14 method for evaluating the seismic resistance of existing buildings. The objective for employing this methodology was to identify weak links that could represent potential seismic hazards. In some cases, the construction documents were incomplete or not detailed enough to draw conclusions. In these cases, additional investigations were recommended.

Table 7.6-1 presents the facilities studied, with their vulnerability to seismic damage, facility priority, upgrade cost, and whether the remedial work can be performed by SWD or an outside contractor.

Table 7.6-1

Summary of Upgrade Recommendations
Cedar Falls Facilities
(Based on 1989 Dollars)

			Estimated Costs								
Facility	Vulnerability[1]	Facility Priority	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Office Shop	Moderate	Medium	$ -0-	$ 300	$ 200	$ 2,500	$ 3,000	$ 200	$ 3,200	±30%	SWD
Sewage Treatment Bldg.	Low	Low	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Well Supply & Pump	Low	Medium	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Totals:			$-0-	$300	$200	$2,500	$3,000	$200	$3,200		

[1] Vulnerability refers to a combined ranking of both the structure and any associated equipment.

8.0 LAKE YOUNGS

8.1 Facility Description

The Lake Youngs facility, located 7 miles southeast of Renton, provides a reservoir storage basin for water received from the Landsburg Diversion. A 96-in. pipeline leaves Lake Youngs and branches into four lines serving the Bellevue and Newport Hills area, West Seattle and Seattle south of the Ship Canal. All water from the Cedar River watershed is routed through the Lake Youngs facility via the Landsburg Diversion.

The Lake Youngs facility is an essential component of the SWD water delivery system. Disruption of the functions of the facility could seriously impact the system's capability to deliver water to a large urban area.

8.2 Geologic Hazard Assessment

Subsurface conditions at Lake Youngs typically consist of glacially consolidated sediments, primarily consisting of glacial till (Zone II on Figure 4.3-1). These sediments, combined with the relatively low topographic relief in the area, generally preclude the development of major earthquake-induced geologic hazards as landslides or liquefaction.

Most of the structures at the Lake Youngs facilities showed signs of satisfactory foundation performance. Some minor hairline cracks were observed during our geological site reconnaissance. These cracks could be attributed to backfill settlement or thermal effects. In any event, these cracks are not suggestive of major foundation distress.

8.3 Office Building

8.3.1 Facility Description

The Lake Youngs Office Building is located directly east of the Lake Youngs Reservoir. The structure measures 40 ft. by 145 ft. and was renovated in 1971. A 40 ft. by 75 ft. portion of the building is used as an office while the remainder is used as a garage and storage area. The building consists of wood framed walls with corrugated metal siding attached. Wood trusses at 2-ft. o.c. form the high-pitched roof. The roof sheathing consists of corrugated metal siding attached to the trusses at 6-in. o.c. The inside of the office area is sheathed with gypsum wall board. The fire wall between the office and the garage is sheathed with two layers of 5/8 in. gypsum wall board each side which extends to the roof diaphragm. The reflected ceiling of the office area is attached to the 1 x 6-in. T & G sheathing attached to the bottom of the trusses.

8.3.2 Structure Evaluations

The original construction documents were not available for review; however, the remodeling drawings were used to gain some information about the structure. The office portion seems better designed to withstand seismic forces than the garage area. At the front elevation of the garage, the structure is mostly open with seven short 2-ft. wall piers. These walls lack sufficient shear strength, sliding resistance, and overturning resistance. Therefore, some damage and large deflections could be expected in the garage area.

The office area offers more seismic resistance than the garage area since the walls are covered with gypsum wall board and the bottom of the trusses have some sheathing on them. Damage should not be as widespread as in the garage area, but some of the walls will be damaged. According to the ATC-14 wood building checklist, this structure should be investigated further to accurately assess the seismic risk involved. Based upon our initial findings, we rate the office building structure as having a moderate vulnerability to seismic damage in a Class I event.

For purposes of cost estimation, our structural upgrade recommendation is to install a bolted stiffening plate across the top of the garage doors.

8.3.3 Equipment and Contents Evaluations

The lockers in the lunch room should be bolted to the walls or the floor to prevent overturning. The racks in the parts room should be bolted. All of the storage parts racks and lockers should be bolted down to the floor slab, with the heavier equipment located on the lower shelf.

8.3.4 Summary and Conclusions

a) From this preliminary structural investigation, we recommend that the building be further investigated to assess the potential seismic hazards. Based upon these preliminary findings, the building should be rated as having a moderate vulnerability to seismic damage in a Class I event.

b) The lockers adjacent to the door in the lunch room portion of the building present a seismic hazard by being adjacent to an exit. These lockers should be anchored. All of the storage racks should be bolted down.

c) Table 8.3-1 summarizes the results of the seismic evaluations. An estimate of remedial upgrade costs is presented in Table 8.3-2.

Table 8.3-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Office Building (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	More analysis required
2	Lockers	2	1	Anchor
3	Storage racks	2	2	Anchor

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 8.3-2

Cost Estimate

Location: Lake Youngs Office Building

Date Constructed: 1971

Priority: Medium

Seismic Strengthening Objective:

1) Improve shear resistance of elevation wall containing garage doors
2) Secure lockers, storage racks

Upgrade Recommendations:

1) Install stiffening plate across top of garage doors
2) Bolt lockers, storage racks to floor and walls

Additional Investigation Required:

1) Detailed evaluation of seismic performance of office building structure

Assumption: Construction Performed In-House

Cost Estimate:

1)	Additional Investigation	$ 3,000
2)	Engineering	4,000
3)	Construction Engineering	2,000
4)	Construction	8,000
	Subtotal:	17,000
5)	Sales Tax (8.1%)	650
	Total:	$17,650

Accuracy of Estimate: +35%

8.4 Equipment Maintenance Building

8.4.1 Facility Description

The Equipment Maintenance Building at Lake Youngs is a light-steel, moment frame building measuring 40 ft. by 80 ft., constructed in 1956. The building eave height is approximately 14 ft. The main function of this building is to provide shelter for and maintain the heavy equipment at Lake Youngs. The building is essentially open at the south elevation with three 18 ft. overhead doors and an additional 12 ft. door. The building has two transverse rigid frames near the center of the building along with one framed and sheathed wall 20 ft. from the west elevation. The rest of the end and side walls consist of metal studs at 16 in. o.c. with exterior metal sheathing.

8.4.2 Structure Evaluations

Roof trusses are spaced at 8 ft. on center between the rigid frames and walls. The roof diaphragm is supported on trusses and rigid frames with tension rods in the form of "K" bracing.

The original construction documents for the metal building were very sketchy; however, with the field inspection, the following observations were made: 1) the building is relatively obsolete by current standards since it combines rigid frames with shear walls; 2) the lateral force resisting system in the transverse direction appears to be very adequate; however, there appears to be a minor torsion problem with the lateral force resisting system in the longitudinal direction since the south elevation is mostly open. If the building materials were not so light, this might be considered a seismic risk. In this case, however, the structure should not suffer major damage during a Class I earthquake. In general, these types of light steel frame buildings have had excellent performance records in past earthquakes. Thus, we rate the building structure as having a low vulnerability to seismic damage in a Class I event.

8.4.3 Equipment and Contents Evaluations

There is little equipment to evaluate; however, the wood cabinet next to the exit should be tied to the building to prevent overturning during an earthquake.

8.4.4 Summary and Conclusions

a) From this preliminary structural investigation, the Equipment Maintenance Building will sustain little damage in a Class I event.

b) The cabinet adjacent to the door in the maintenance portion of the building presents a seismic hazard by being adjacent to an exit. This cabinet should be tied to the wall.

c) Table 8.4-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 8.4-2.

Table 8.4-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Equipment Maintenance Building (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Wood cabinet	3	2	Tie to wall

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 8.4-2

Cost Estimate

Location: Lake Youngs Equipment Maintenance Building

Date Constructed: 1956

Priority: Low

Seismic Strengthening Objective:

1) Prevent lockers and cabinets from overturning

Upgrade Recommendations:

1) Bolt lockers and cabinet to wall or floor

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 0
2)	Construction Engineering	0
3)	Construction	100
	Subtotal:	100
4)	Sales Tax (8.1%)	10
	Total:	$ 110

Accuracy of Estimate: ±30%

8.5 Storage Building

8.5.1 Facility Description

The storage building at Lake Youngs is a light steel moment frame building measuring 32 ft. by 38 ft. It is located west of the equipment and maintenance building. The building eave height is approximately 11 ft. The main function of this building is to provide storage space. This building consists of three transverse rigid frames with purlins spanning between the frames to support the roof sheathing. Attachment of the roof sheathing to the roof purlins appears to be made with wires at about 8 in. on center. One 8 ft. door opening exists at the east transverse wall.

8.5.2 Structure Evaluations

The original construction documents for the metal building were unavailable; however, from the field inspection the structure appears to conform to normal industry design practices for a building of its size, except for the connection of the roof sheathing to the roof purlins. In general, light steel frame buildings such as this have had excellent performance records in past earthquakes. Therefore, we rate this structure as having a low vulnerability to seismic damage in a Class I event.

8.5.3 Equipment and Contents Evaluations

There are no major equipment items in the storage building.

8.5.4 Summary and Conclusions

a) From this preliminary structural investigation, the Storage Building will adequately withstand a Class I earthquake event.

b) Table 8.5-1 summarizes the results of the seismic evaluations.

Table 8.5-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Storage Building (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

8.6 Pipe Storage Area

8.6.1 Facility Description

The main storage area at Lake Youngs is a clearing upon which concrete rails with 2 x 4-in. sills have been built to serve as racks for pipe storage. Ductile iron pipe of all sizes is stored on the rails for use in construction and repair of pipelines. Most of this replacement pipe stock is old and rusted. The racks are built to enable stacking and removing pipe with fork lifts.

Pipes are stacked four and five high in the 18-in. and 12-in. sizes, each layer separated by 2 x 4-in. boards. Wood wedges are generally used to prevent the pipe from rolling off the boards. The pipes are not strapped on the racks.

There is also a new pipe storage area along a side road at the end of the lake. In this area, the pipe sections are stored end-to-end without stacking.

8.6.2 Structure Evaluations

For the main pipe storage yard, the wedges, where used, are not adequate to prevent pipe from rolling off the racks in the event of an earthquake. Collapse of the stacks can be expected unless additional measures are taken to restrain the pipe.

It is difficult to assess how much damage might be done to the pipe if the stacks were to collapse. Industry experience indicates that cracked bell housings may result when pipes are dumped from trucks onto other pipe, particularly when the material is cast iron as opposed to ductile iron. Thus, it is reasonable to conclude that some cracking would result. Given the possibility that the pipe would be needed to make emergency repairs following an earthquake, the risk of cracked pipe should be minimized.

We recommend that the pipe stack height be minimized as much as possible. Pipe should not be stored in stacks more than four in height. Where possible, stacks should be kept to two in height.

A second measure to prevent stack collapse is to strap the pipe in the racks. Metal band straps or nylon straps with end buckles are recommended.

For the new pipe storage area, because the pipe sections were not stacked, this area was judged to have a low seismic vulnerability.

8.6.3 Summary and Conclusions

The pipes stored in the Lake Youngs pipe storage area are subject to damage from the collapse of the stacks. Stack height should be minimized, with the total height not to exceed four pipes. In addition, it is recommended that the pipe in the stacks be strapped together.

An estimate of remedial upgrade costs is presented in Table 8.6-1.

Table 8.6-1

Cost Estimate

Location: Lake Youngs Pipe Storage Area

Priority: High

Seismic Strengthening Objective:

1) Prevent pipe from rolling off stacks
2) Prevent pipe from falling from height

Upgrade Recommendations:

1) Strap the pipe in the stacks with metal bands or nylon straps.

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 1,500
2)	Construction Engineering	500
3)	Construction	3,000
	Subtotal:	5,000
4)	Sales Tax (8.1%)	250
	Total:	$5,250

Accuracy of Estimate: ±25%

8.7 Chlorination Building

8.7.1 Facility Description

The Lake Youngs Chlorination building was constructed in 1977 and treats the water from the Lake Youngs Storage Reservoir. The building is located at the north end of the reservoir and consists of a reinforced pumice block structure. The building is an "L" shaped structure, 30-ft. wide and 60-ft. long at the longer axis, and 25-ft. wide by 30-ft. long on the shorter axis. The roof structure consists of 3-in. T & G decking spanning between glu-lam beams that support the roof structure. The beams spanning the width of the building are typically supported on reinforced wall pilasters with an intermediate block or concrete pilaster. The beams are supported on a variety of block pilasters with the minimum being a 16 x 16-in. reinforced block pilaster. Typical reinforcement for the pilasters are #5 vertical and #3 horizontal ties at 10 in. on center. The taller portion of the building utilizes intermediate 12 sq. in. reinforced concrete columns to support the roof beams.

A new addition is of steel frame construction with 8-in. concrete mortar units. It measures 31 ft. by 82 ft. and supports a crane with a 3-ton hoist. It has a standing seam metal roof with a 4-on-12 pitch. There is a 10-in. thick continuous concrete perimeter foundation tied to a 6-in. slab on grade. This addition houses the chlorine tank racks.

8.7.2 Structure Evaluations

According to the original construction drawings, the block walls are 8-in. thick and are reinforced horizontally with "K-web" joint reinforcing at every third course. No vertical wall reinforcing was called for in the plans except the four #5 vertical bars with #3 ties at each block pilaster and the trim vertical bars typically at door and window penetrations. The roof T & G diaphragm stresses are expected to be low since the glu-lam beams provide a continuous tie across the building in the transverse direction. This continuous tie serves to divide the two larger roof diaphragms into smaller diaphragms formed between the glu-lam beam lines. The upper walls of this building are tied together with a continuous bond beam through the block pilasters containing two #5 bars.

The block wall-to-roof diaphragm connection typically consists of a T & G attachment to a top plate bolted to the top of the block wall. This connection would appear to stress the wood top plate across the grain, which is no longer permitted by the current code. However, if the out-of-plane seismic wall forces in the span between the reinforced wall pilasters were to be analyzed, this cross grain tension problem would most likely be found to be insignificant. We recommend that this problem be investigated further, in conjunction with the planned facility enlargement project.

Shear capacity of the masonry walls does not appear to be critical, with the possible exception of the southwest elevation, due to the large door penetrations. This should be investigated further to determine the actual shear capacity and amount of damage to be expected. The foundation of the structure is adequately reinforced and is not a major concern. According to the ATC-14 reinforced masonry checklist, this structure has some weak links, mainly due to lack of vertical reinforcing and positive connection of the masonry walls to the roof diaphragm. Thus, the structure should be analyzed further to assess the potential seismic risk. We rate this structure as having a moderate vulnerability to seismic damage in a Class I event, based upon the problems identified from this preliminary investigation. Nonetheless, we believe that the building would remain functional and most probably no strengthening will be required.

In the plans for the new addition, there appears to be adequate vertical and horizontal reinforcing and anchorage of the block walls. The top of the masonry wall is well connected to the steel frame columns with continuous angle steel and anchor bolts at 4-ft. o.c. The latter are embedded in the top two courses of masonry, which are reinforced bond beams. The roof is well-connected to the walls with 16-gauge continuous channels welded to the roof purlins at the end walls and bolted and welded plates and channels connected to the block walls and steel frame at the side walls.

Given the positive connections and reinforcement as discussed above, the new addition as shown in the plans is well-detailed and will perform satisfactorily in a Class I earthquake.

8.7.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This equipment is located near the center of the building. The chlorinator cabinet is framed out of light-weight plastic and is anchored at all four corners. Most of the piping connected to the equipment is plastic, which has some flexibility for movement. The steel piping, however, is not that flexible and could rupture during an earthquake. The reliability of this equipment could be improved by providing flexible couplings on the piping. This equipment has low vulnerability to seismic damage.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method of solving this potential hazard is to strap the chlorine tanks

around their center. The strap should have an easy de-coupling connector to facilitate moving the tanks. An alternative solution is to construct removable reinforced bollards at the ends of the tank rack, and tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

Miscellaneous Equipment

The chlorine control panel in the control room is properly anchored to both the glu-lam beams at the top and to the concrete slab. The control and alarm unit, called the Betalarm, should be anchored to the floor slab.

8.7.4 Summary and Conclusions

a) From this preliminary structural investigation, we have identified several possible weak links that need to be investigated further. After a more in depth engineering analysis of some of the items listed in the structure section, some remedial strengthening of the building may be required.

b) Most of the equipment within the chlorination building has a low vulnerability to seismic damage. The chlorine tanks should be strapped together or braced as specified above.

c) Table 8.7-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 8.7-2.

Table 8.7-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Chlorination Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	More analysis
2	Chlorinators	1	1	Consider flexible couplings
3	Chlorine tanks	2	1	Provide straps

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 8.7-2

Cost Estimate

Location: Lake Youngs Chlorination Building

Date Constructed: 1977

Priority: High

Seismic Strengthening Objectives:

1) Prevent chlorine tanks from sliding, rupturing feeder lines.

Upgrade Recommendations:

1) Secure chlorine tanks with strap and ratchet arrangement, or construct removable reinforced bollards at the end of the tank rack and install tube steel rails paralleling the tank rack on both sides

Additional Investigation Required:

1) Detailed evaluation of seismic performance of chlorine building structure

Assumption: Construction Performed In-House

Cost Estimate:

1)	Additional Investigation	$4,000
2)	Engineering	$ 1,700
3)	Construction Engineering	100
4)	Construction	1,700
	Subtotal:	7,500
5)	Sales Tax (8.1%)	150
	Total:	$7,650

Accuracy of Estimate: ±30%

8.8 Corrosion Treatment Building

8.8.1 Facility Description

The Lake Youngs Corrosion Facility building was constructed in 1980 and injects a chemical (lime) into the pipelines supplied from the Lake Youngs Reservoir to make the water non-corrosive. The facility is located north of the Lake Youngs Reservoir and east of the Lake Youngs Chlorination Building. The structure measures 22 ft. by 40 ft. and is approximately 14-ft. tall. The lime storage bin extends about 30 ft. above the roof structure of the building and is about 15-ft. in diameter.

8.8.2 Structure Evaluations

The foundation elements of the building appear to be adequately reinforced and have a low vulnerability to seismic damage. The walls of the structure are 8-in. reinforced, fully grouted c.m.u. The reinforcing provided in the block walls is consistent with the current UBC minimum requirements. The roof of the building is constructed of 2 x 8-in. roof joists at 24-in. o.c. with 3/4-in. plywood attached.

The lime storage bin is supported on four wide flange steel columns anchor bolted to the concrete foundation. The support of the storage bin appears to have been designed by the manufacturer to resist seismic forces. According to the original construction drawings, the block walls are fully grouted and adequately reinforced to provide a sufficient amount of in-plane shear capacity during an earthquake. This structure rated adequately when evaluated with the ATC-14 reinforced masonry checklist with one exception. The penetration in the roof diaphragm for the storage bin is large and could possibly limit the available perpendicular-to-wall bracing provided by the wood diaphragm. This should be analyzed further to determine if this poses a major seismic risk. This structure was recently constructed and has current seismic design features that provide acceptable building performance. Thus, we rate this structure as having a low vulnerability to seismic damage.

8.8.3 Equipment and Contents Evaluations

Computer Panels

The computer room control panel labelled "Y" needs to be bolted to the floor.

Electrical Panels

The electrical panels and piping appear to be properly anchored.

8.8.4 Summary and Conclusions

a) From this preliminary structural investigation, we have identified a possible weak link due to the large roof diaphragm opening for the storage bin. A more in-depth engineering analysis of this particular area may indicate that some strengthening of the diaphragm is required. However, based upon the findings of this preliminary investigation, the Corrosion Facility structure is rated as having a low vulnerability to seismic damage, and remedial upgrades are not anticipated.

b) The equipment within the treatment building has a low vulnerability to seismic damage, with the exception of the computer control panel "Y" which needs to be anchored.

c) Table 8.8-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 8.8-2.

Table 8.8-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Corrosion Treatment Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	More analysis required
2	Equipment	2	1	Anchor computer panel "Y"

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Note: This facility is high priority because it houses the emergency power generator.

Table 8.8-2

Cost Estimate

Location: Lake Youngs Corrosion Treatment Building

Date Constructed: 1980

Priority: High

Seismic Strengthening Objectives:

1) Secure computer control panel "Y".

Upgrade Recommendations:

1) Anchor panel "Y" to floor or wall with concrete expansion anchors.

Additional Investigation Required:

1) Evaluate effect of large penetration in roof diaphragm

Assumption: Construction Performed In-House

Cost Estimate:

1)	Additional Investigation	\$3,100
2)	Engineering	1,100
3)	Construction Engineering	0
4)	Construction	500
	Subtotal:	4,700
5)	Sales Tax (8.1%)	40
	Total:	\$4,740

Accuracy of Estimate: ±20%

8.9 Chlorine Residual Analyzer Building

8.9.1 Facility Description

This facility is located within the Lake Youngs Reservation just north of the corrosion facility. Its function is to measure the amount of chlorine in the water flowing from the Lake Youngs reservoir to the chlorination building and into the water system. The building is of single story, pumice block-and-mortar construction, measuring 7 ft. by 9 ft., and about 8 ft. high. The construction drawings were not available; however, the construction of this building is probably very similar to the Sewage Treatment and Chlorine Building at Cedar Falls, with the exception of the roof construction. The building has only one 3-ft. wide door opening and is not critical in shear.

8.9.2 Structure Evaluations

Typical construction of these types of buildings consists of 8-in. pumice block without any reinforcing except probably around the door lintel. The roof is constructed from 2 x 6-in. joists sloped for drainage with an unknown attachment to the roof wood plate above the pumice block. If the ATC-14 rapid evaluation methodology was rigorously applied, it would point out several weak links in the structure due mainly to the lack of reinforcing. However, since this building is small and light weight it will not experience a large amount of seismic acceleration or major damage, even though it is essentially unreinforced.

With these considerations, this structure is judged to have a low vulnerability to a Class I earthquake.

8.9.3 Equipment and Contents Evaluations

The equipment in this building consists of a chlorine analyzer, turbidimeter, and a few carbon dioxide bottles that are chained to the wall. All equipment is well anchored. The chlorine analyzer unit is provided with flexible couplings and has a low vulnerability to seismic damage.

8.9.4 Summary and Conclusions

a) Based upon this preliminary investigation, we do not anticipate a need for seismic strengthening of this building, as it is small and will experience little damage from a Class I earthquake.

b) All equipment is well-anchored and has a low vulnerability to damage.

c) Table 8.9-1 summarizes the results of the seismic evaluations.

Table 8.9-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Chlorine Residual Analyzer Building (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Unreinforced
2	Equipment	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

8.10 Valve Vault

8.10.1 Facility Description

The proposed Lake Youngs Valve Vault is expected to be in operation by 1991. The structure will consist of a nearly completely below-grade, cast-in-place reinforced concrete building on a mat foundation. The building measures 15-ft. wide by approximately 26-ft. long. The concrete slab roof is supported by transverse and longitudinal concrete beams adjoining two large roof hatch openings and is nearly 7 ft. above the intermediate floor grating. The total structure height is approximately 17 ft..

8.10.2 Structure Evaluations

From the proposed construction documents the concrete perimeter walls are 15-in. thick and have #6 bars at 9-in. o.c. each face each way, which exceeds the requirements of the current ACI code. The connection of the walls to the slab diaphragm are adequate to resist the Class I seismic forces. The walls are adequately doweled to the mat foundation. The access openings in the roof diaphragm appear to be adequately reinforced to transfer the lateral load through the opening and eventually into the shear walls. When the ATC seismic evaluation sheet was filled out, no weak links were discovered with the structural details of this proposed structure. In conclusion, it is our opinion that in a Class I earthquake this structure will experience only minor damage.

8.10.3 Equipment and Contents Evaluations

Since this structure is not yet built, no evaluation is provided for the contents of the building.

8.10.4 Summary and Conclusions

a) From the preliminary investigation of the proposed construction documents, it appears that this structure will experience only minor structural damage. In our opinion, no need for revision of the structural details is required.

Table 8.10-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Valve Vault (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

8.11 Valve Well Building

8.11.1 Facility Description

The Lake Youngs Valve Well Building is located directly north of the Lake Youngs Reservoir. The structure measures 22 ft. by 15 ft. and was constructed in 1929. The building is constructed of 8-in. concrete walls extending to about 14 ft. above the floor level. The original window openings have been filled in with concrete. The roof is a concrete slab supported by the exterior walls and an intermediate steel beam spanning the long direction of the building. The beam is supported on concrete wall pilasters within the transverse walls. The Valve Well Building is centered over the valve well structure.

8.11.2 Structure Evaluations

The valve well structure consists of 22-in. thick concrete walls that extend to approximately 55 ft. below the surface of the ground. The structure is not reinforced according to the original drawings, dated approximately 1926. Considering the shear mass of this structure and its circular shape, we estimate that in a Class I seismic event, the damage to this structure will be minor.

8.11.3 Equipment and Contents Evaluations

There is little equipment associated with this structure. The electrical equipment and valves appear to be well secured and have a low vulnerability to damage.

8.11.4 Summary and Conclusions

a) From this preliminary structural investigation of the Valve Well Building, it is expected that the structure will adequately resist a Class I seismic event.

b) The equipment associated with the valve well structure appears well-secured.

c) Table 8.11-1 summarizes the results of the seismic evaluations.

Table 8.11-1

Summary of Preliminary Seismic Evaluations
Lake Youngs Valve Well Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

8.12 Summary and Conclusions

8.12.1 Geotechnical

There does not appear to be any major geologic hazards that would threaten the earthquake functionality of the Lake Youngs facilities. This conclusion is based upon a limited site review of the existing facilities.

8.12.2 Structures and Equipment

The evaluation of the Lake Youngs facilities was based on a site inspection, a review of construction drawings, and an evaluation prepared with the assistance of the ATC-14 method for evaluating the seismic resistance of existing buildings. The objectives for employing this methodology was to identify weak links that could represent potential seismic hazards. In many cases, the construction documents were incomplete or not detailed enough to draw conclusions. In these cases, additional investigations were recommended.

Table 8.12-1 presents the facilities studied, with their vulnerability to seismic damage, upgrading priority, projected cost, and whether the SWD can perform the upgrade in-house or with an outside contractor.

Table 8.12-1

Summary of Upgrade Recommendations
Lake Youngs Facilities
(Based on 1989 Dollars)

Facility	Vulnerability[1]	Facility Priority	Estimated Costs: Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Office Building	Moderate	Medium	$ 3,000	$ 4,000	$ 2,000	$ 8,000	$ 17,000	$ 650	$ 17,650	±35%	SWD
Maintenance Building	Low	Low	-0-	-0-	-0-	100	100	10	110	±30%	SWD
Storage Building	Low	Low	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Pipe Storage Area	Moderate	High	-0-	1,500	500	3,000	5,000	250	5,250	±25%	SWD
Chlorination Building	Moderate	High	4,000	1,700	100	1,700	7,500	150	7,650	±30%	SWD
Corrosion Facility	Low	High	3,100	1,100	-0-	500	4,700	40	4,740	±20%	SWD
Chlorine Residual Bldg.	Low	Medium	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Valve Vault	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Valve Well Building	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Totals:			$10,100	$8,300	$2,600	$13,300	$34,300	$1,100	$35,400		

(1) Vulnerability refers to a combined ranking of both the structure and any associated equipment.

9.0 CONTROL WORKS

9.1 Facility Description

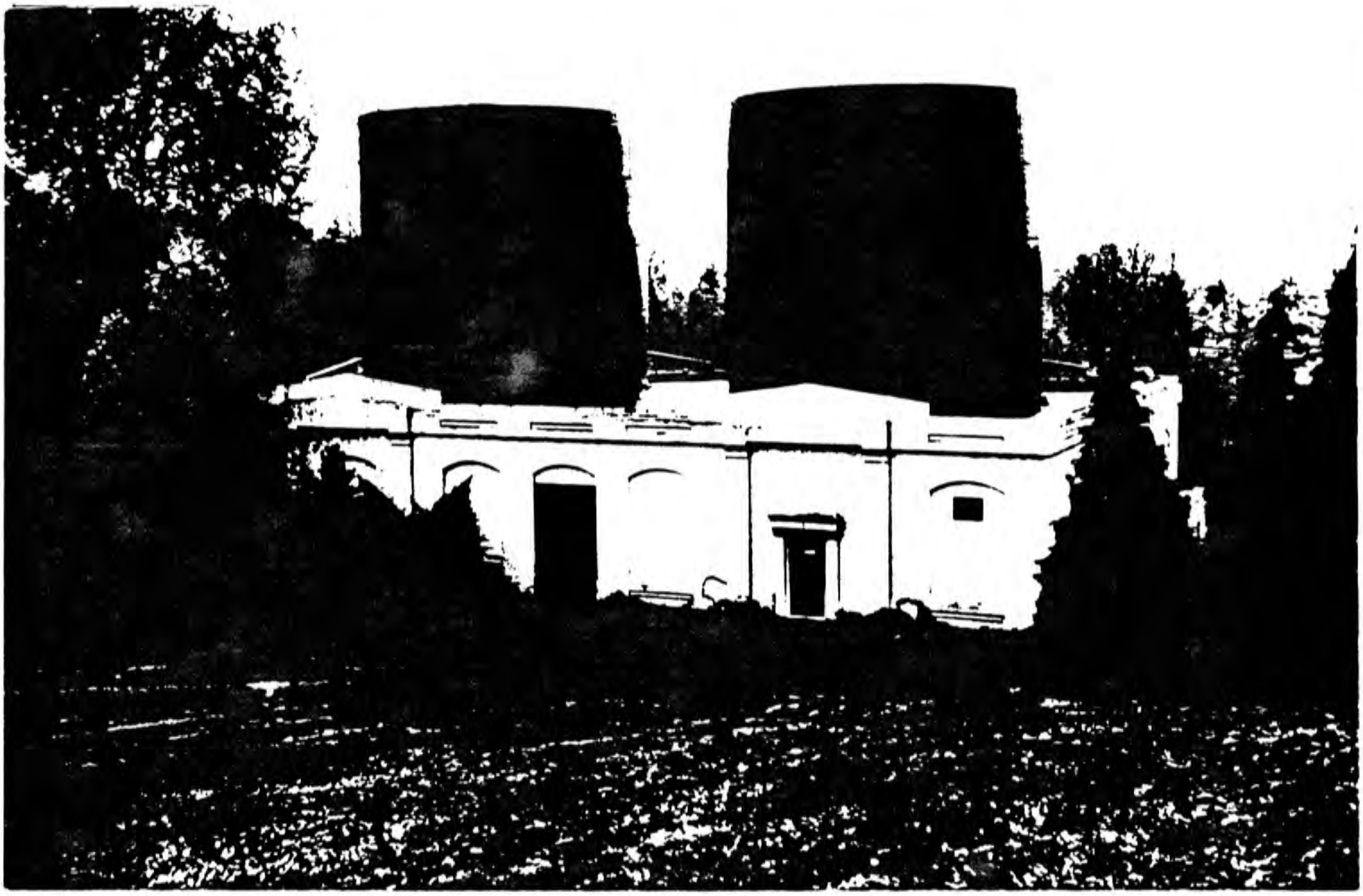
The Control Works Facility, as shown in Figure 9.1-1, is located two miles east of Lake Youngs. It connects the 96-in. diameter Lake Youngs tunnel to the Cedar River pipelines. The tunnel splits at the building wall into a 66-in. bypass line and two short 66-in. lines which feed the surge tanks. These two 30-ft. diameter surge tanks are the dominating features in and above the building. They are 65-ft. tall, extending from the basement floor to 29 ft. above the roof. The surge tank outlets and the bypass feed the 72-in. lake header, which in turn feeds the 60-in. to 72-in. Cedar River pipelines. A 66-in. river header connects the 78-in. Lake Youngs bypass to the Cedar River pipelines also.

The structure, built in 1923, is a cast-in-place concrete building with plan dimensions of 60 ft. x 84 ft, and is shown in Figures 9.1-2 to 9.1-11. The first story is 20 ft. high and it is totally buried on the uphill (southeast) side and 11 ft. buried on the northwest side. The basement walls below-grade are thick concrete gravity walls. The center line of the pipe is 7 ft. above the basement floor.

A 20 ft. x 84 ft., one-story concrete addition is situated on the northwest side of the main structure. This houses the river header.

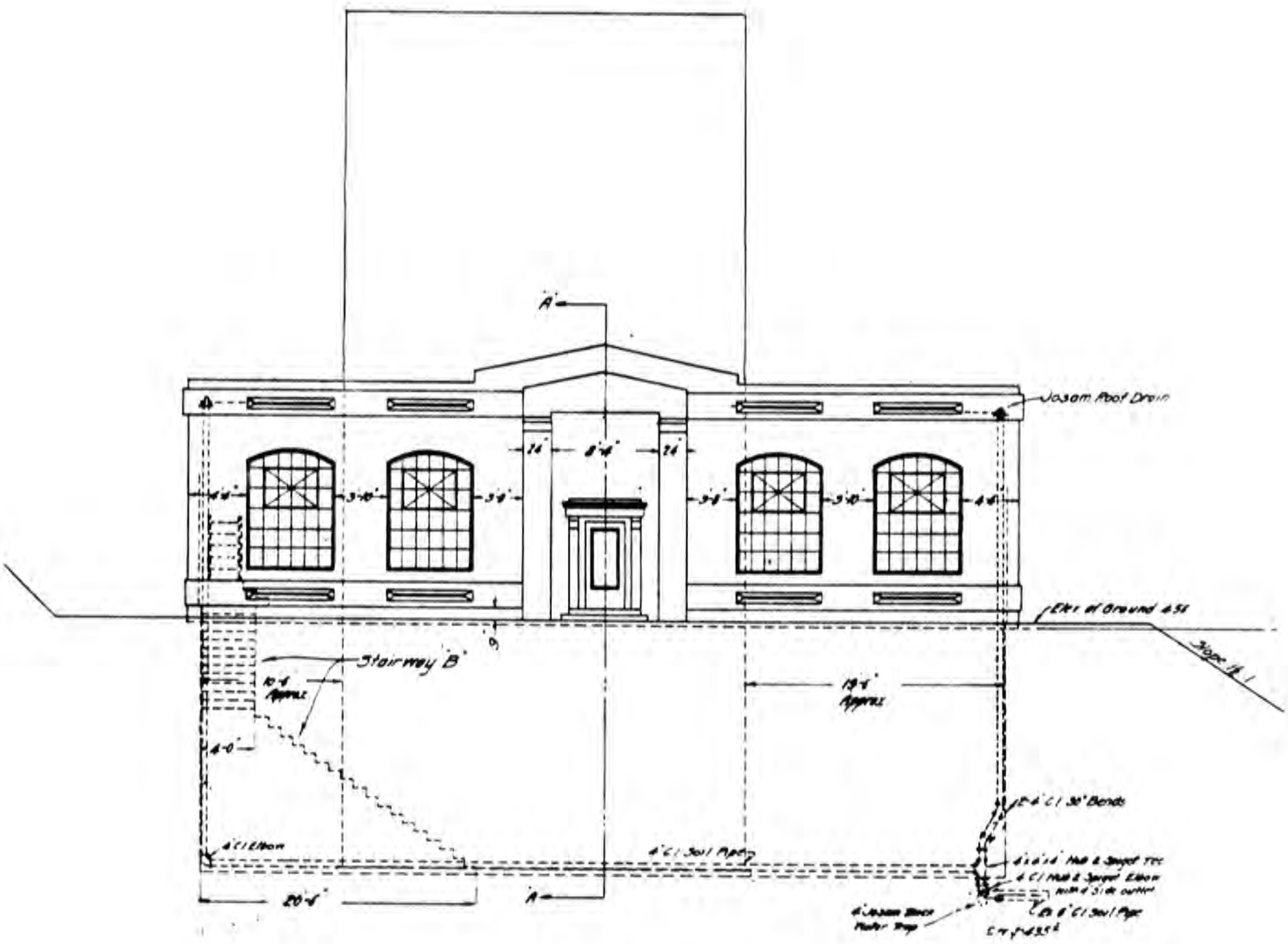
The roof is 16 ft. above the first floor. It is formed by a 4-in. concrete slab which spans between a series of beams and small columns. The second floor is a 5-in. thick slab with similar framing to the roof. Both these slabs are separated from the surge tanks by a 1/4-in expansion joint with filler material.

The exterior walls above-grade are 16 in. thick. They have a series of windows which have all been infilled with unreinforced concrete blocks. The tanks and pipes are constructed of riveted steel.



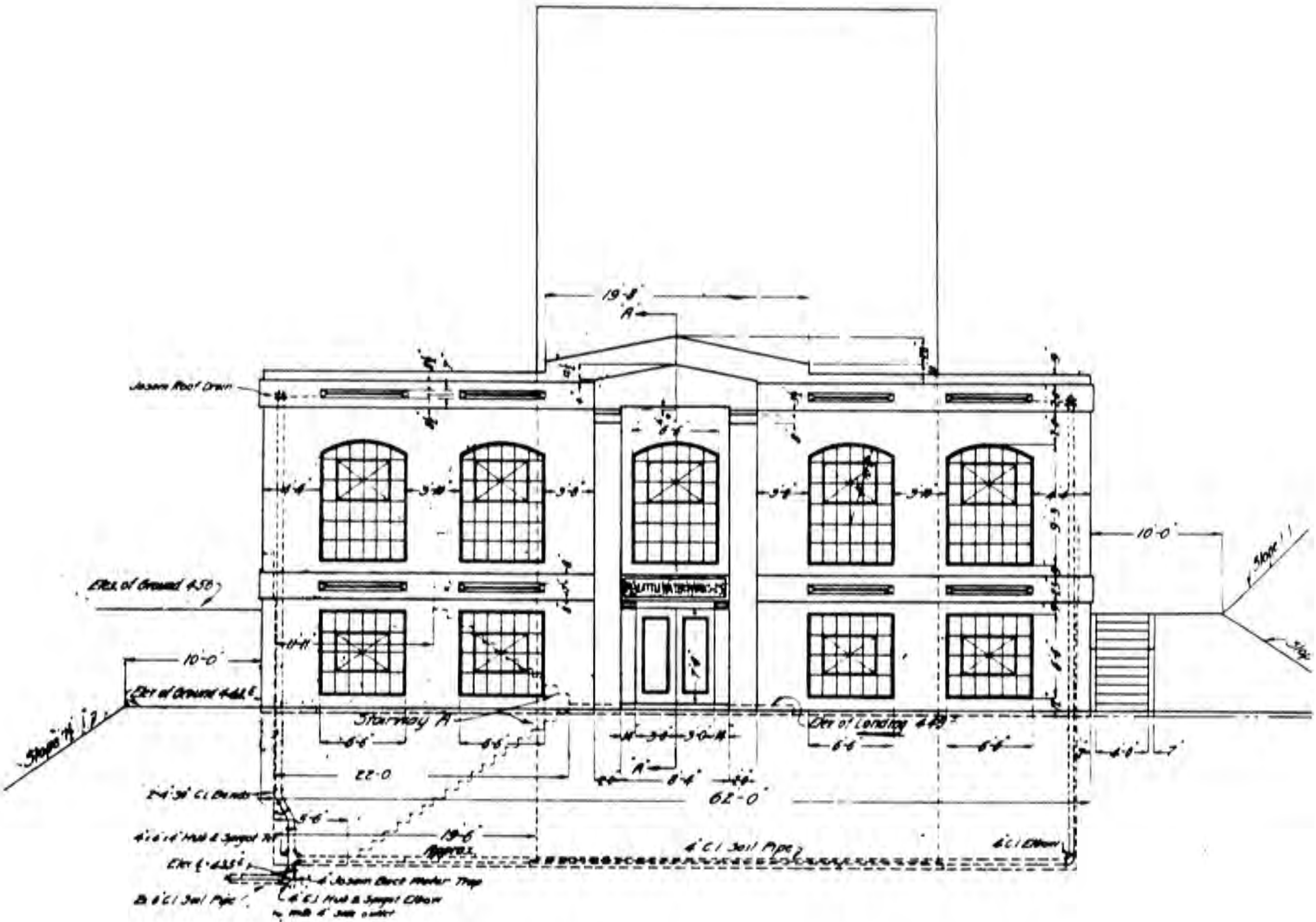
Control Works Facility

Figure 9.1-1



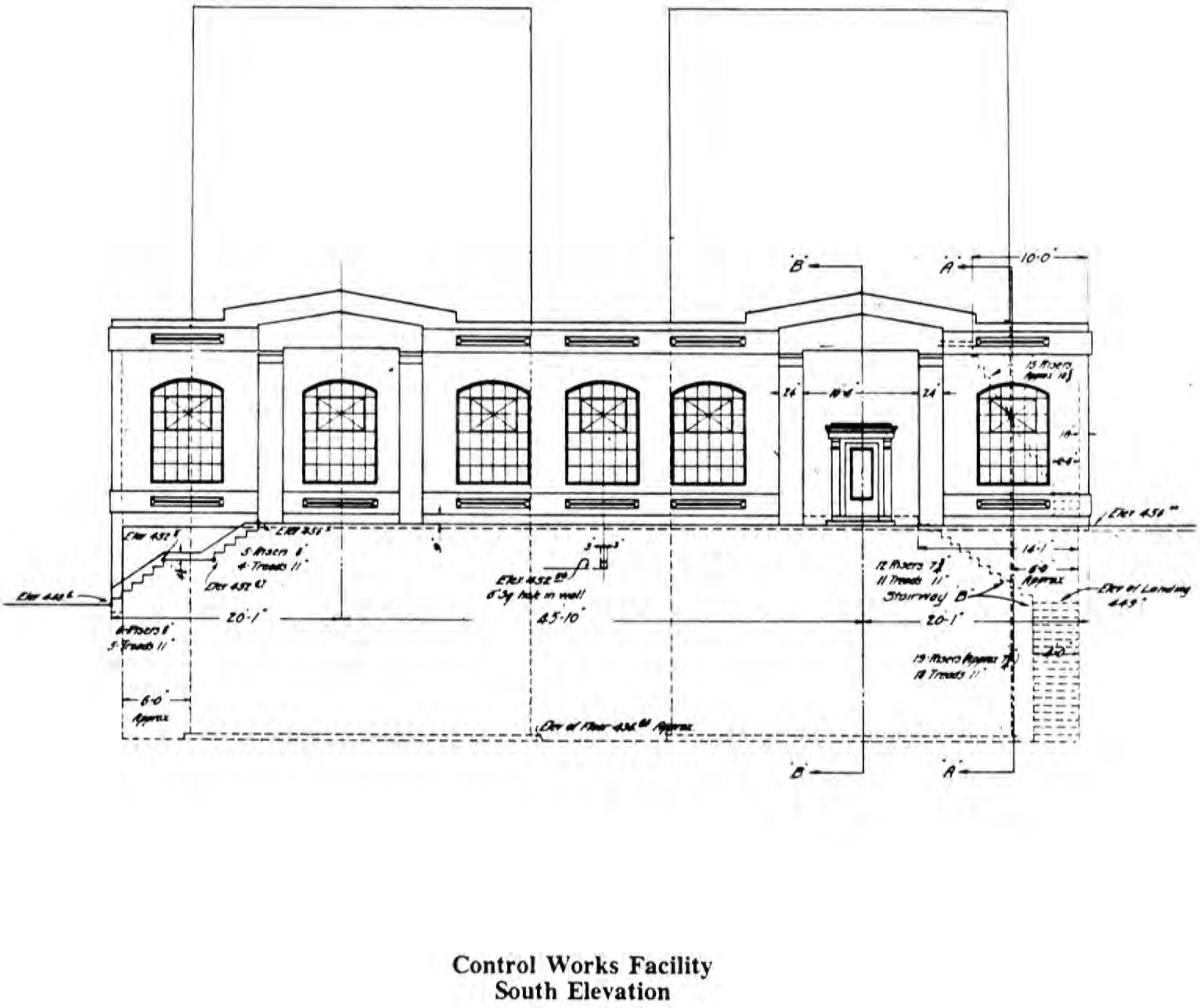
Control Works Facility
East Elevation

Figure 9.1-2



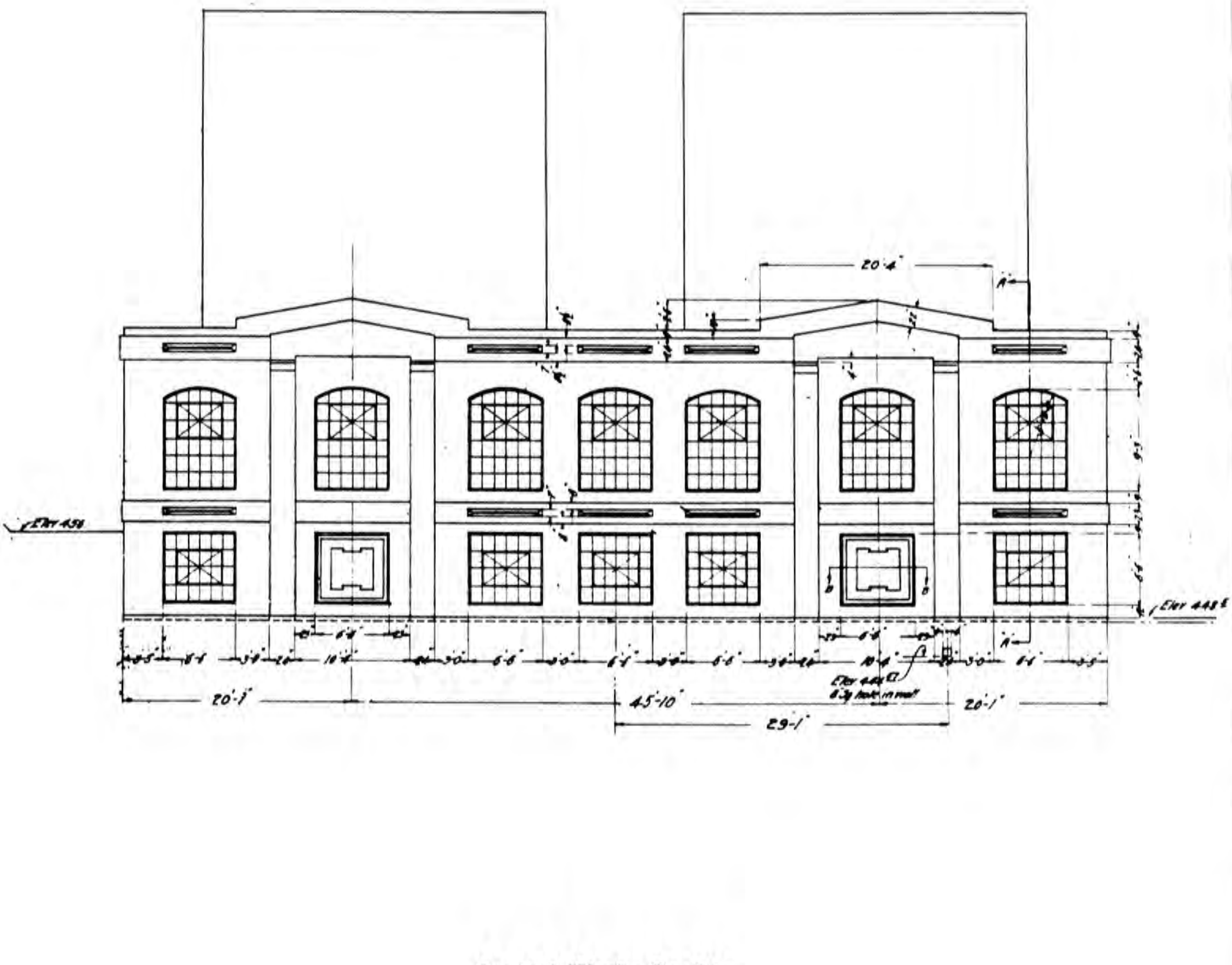
Control Works Facility
West Elevation

Figure 9.1-3



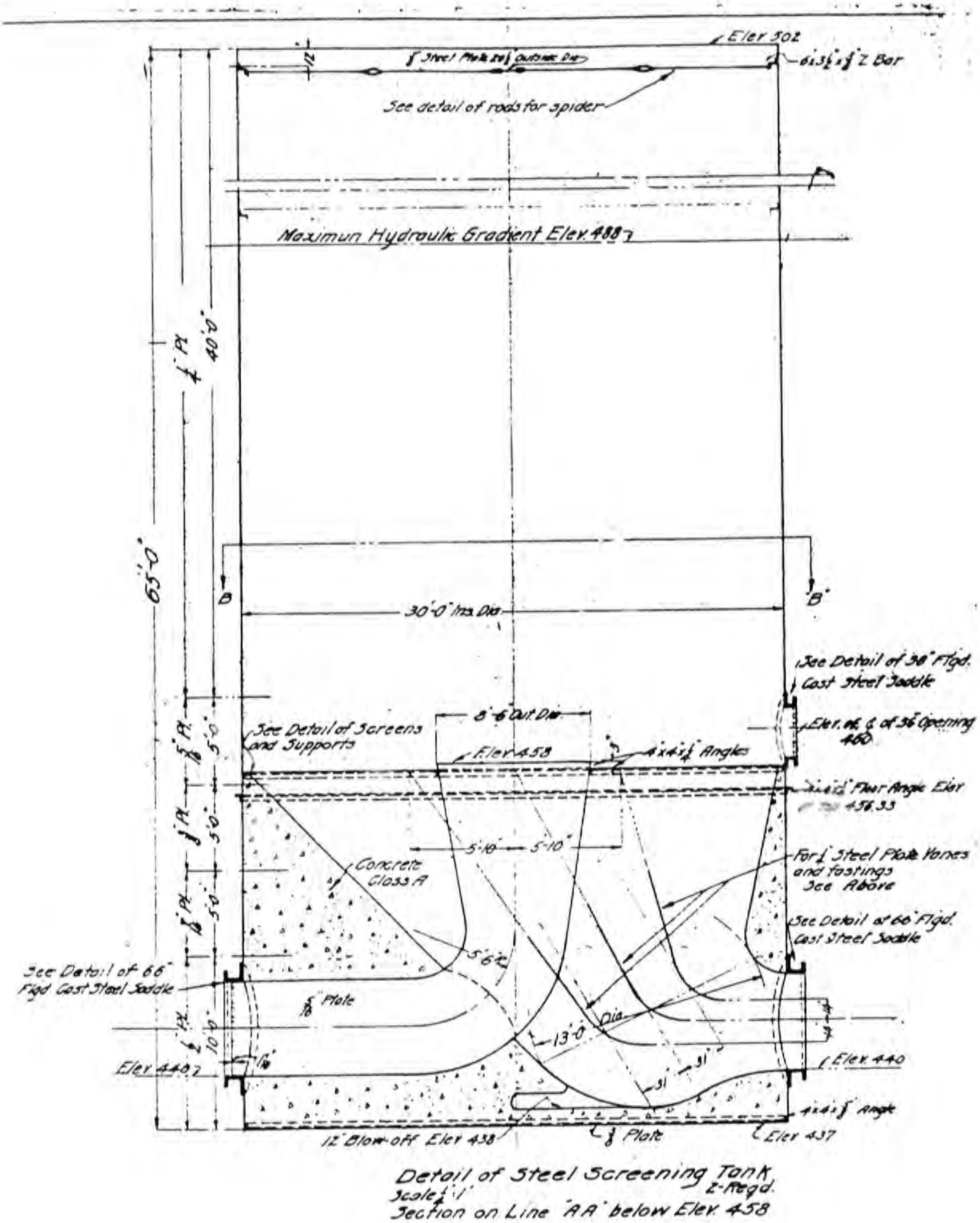
Control Works Facility
South Elevation

Figure 9.1-4



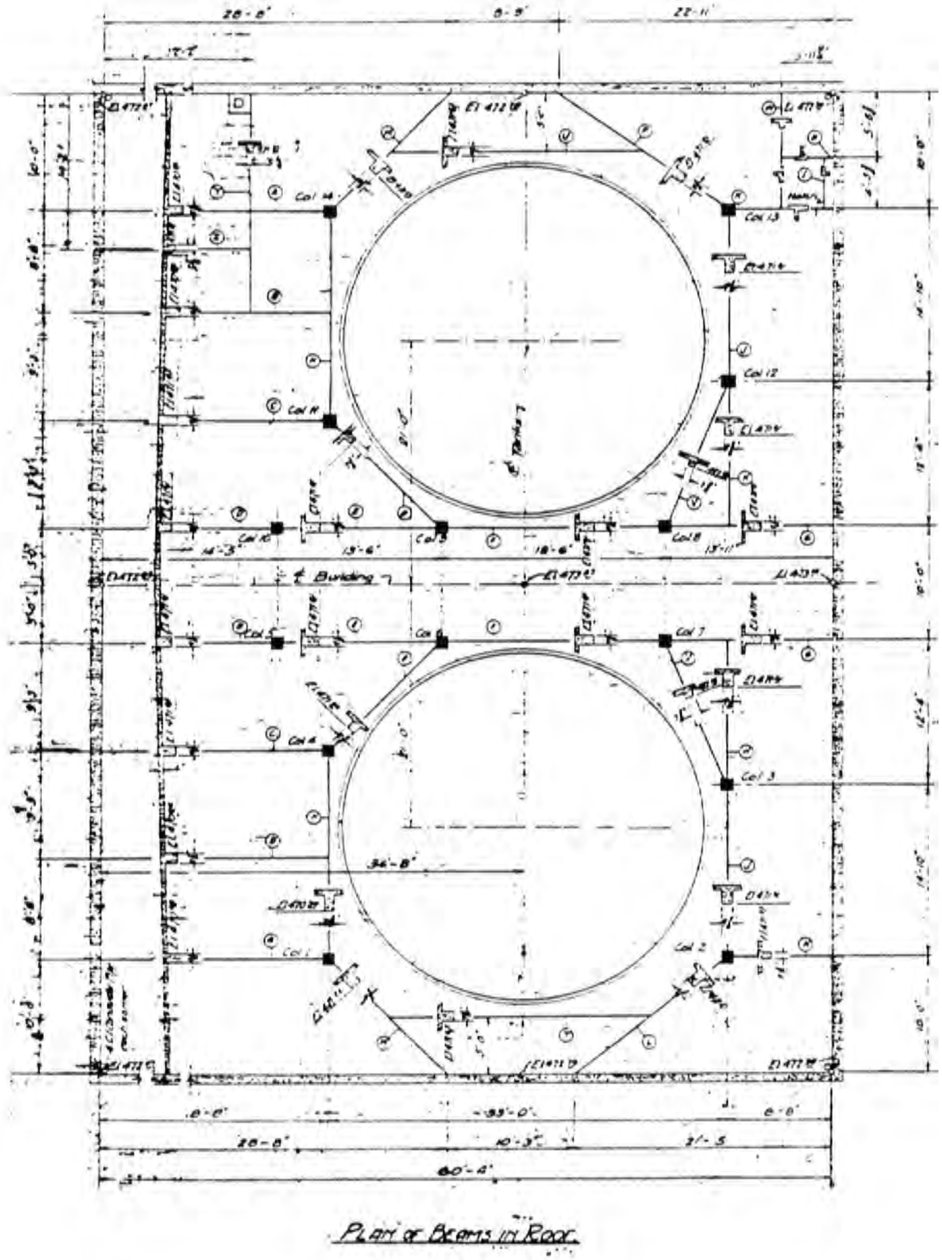
Control Works Facility
North Elevation

Figure 9.1-5



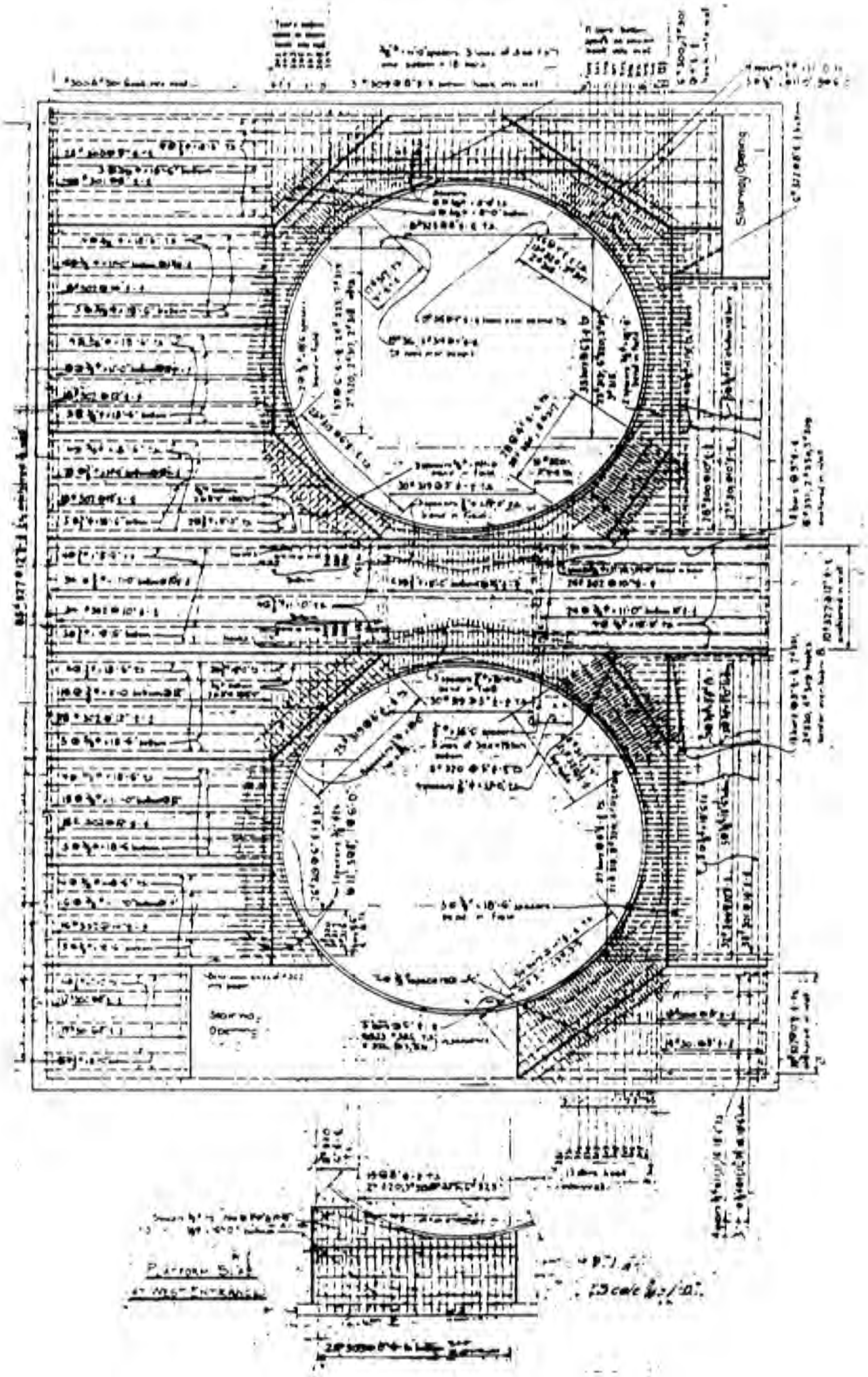
Control Works Facility
Standpipe Cross-Section

Figure 9.1-6



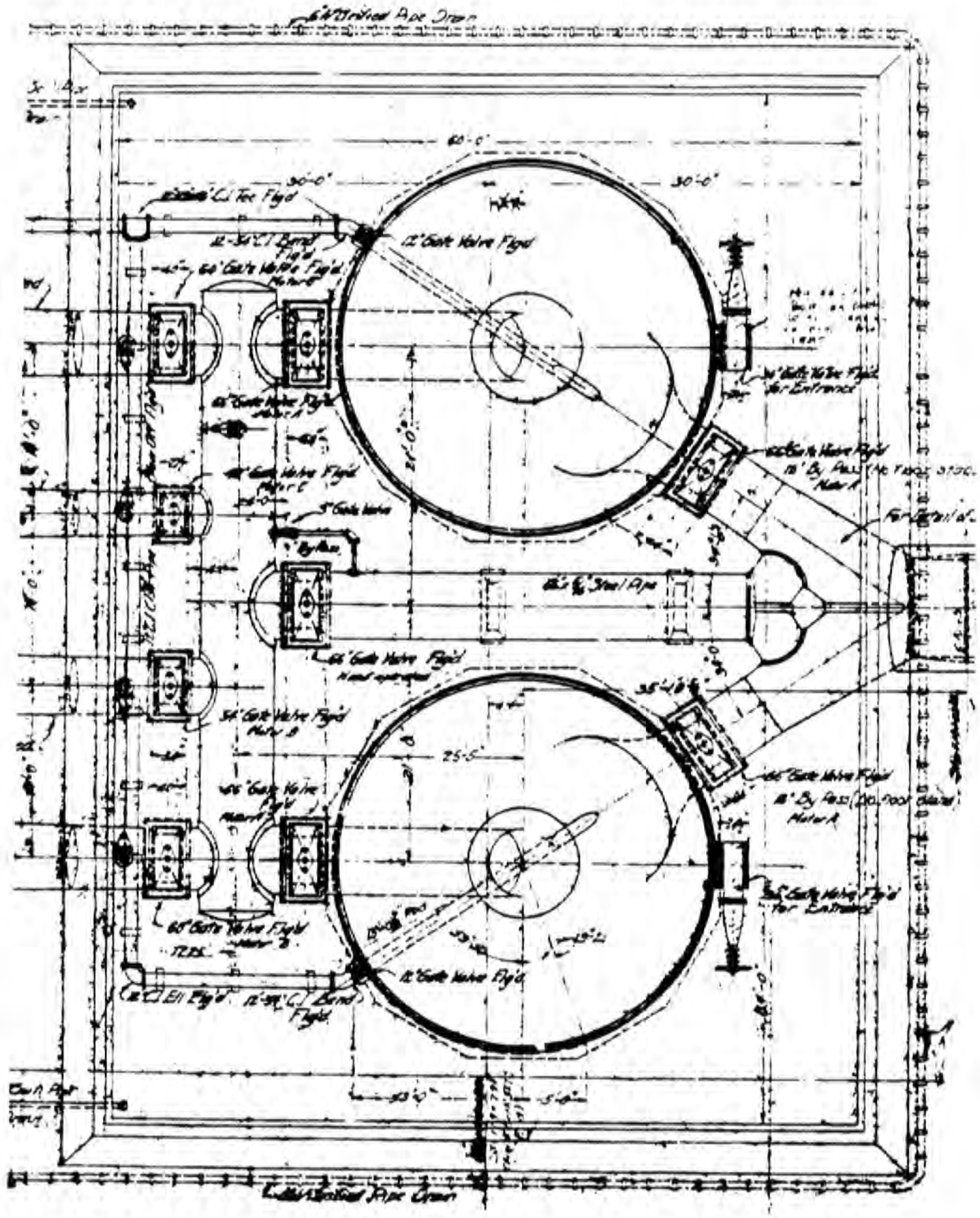
Control Works Facility
Roof Plan

Figure 9.1-7



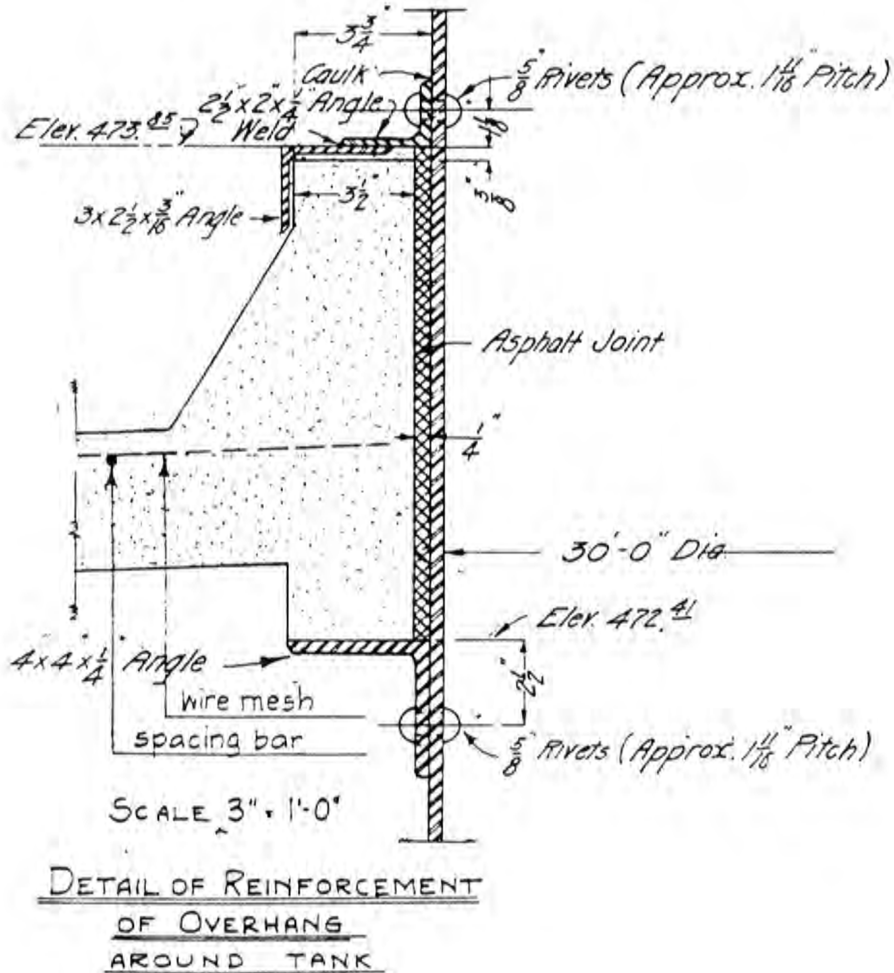
Control Works Facility
Slab Reinforcement

Figure 9.1-8



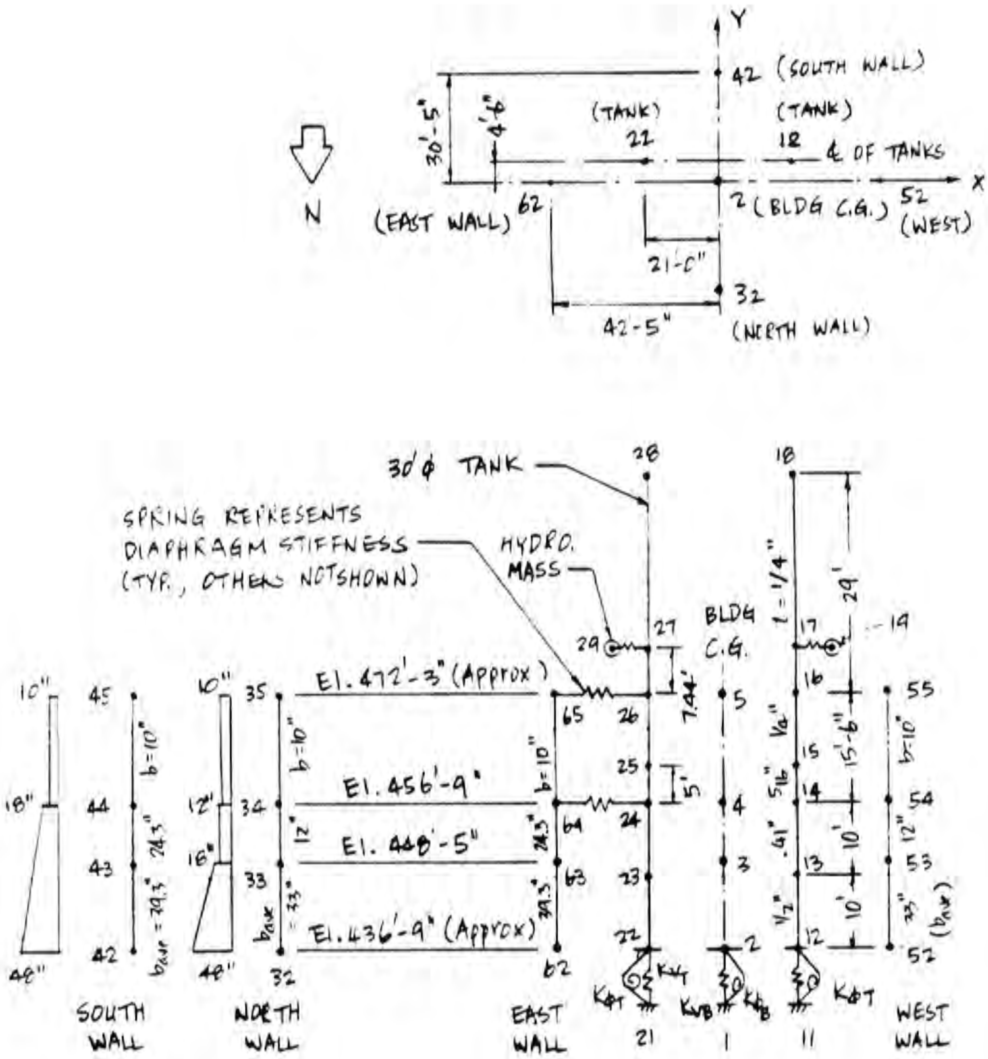
Control Works Facility
First Floor Plan

Figure 9.1-9



Control Works Facility
Expansion Gap Detail

Figure 9.1-10



Control Works Facility
Mathematical Model

Figure 9.1-11

9.2 Geologic Hazard Assessment

The Control Works building is underlain by glacial till as shown on regional geological maps (Zone II on Figure 4.3-1). The combination of this competent foundation material and the relatively minor slopes in the vicinity of the Control Works building would typically preclude development of liquefaction or major earthquake-induced landsliding adjacent to the structure.

Some minor settlements were observed in the fill surrounding the Control Works building. As it is anticipated that the building is founded upon competent, native soils, settlement of the backfill would generally be inconsequential to the performance of the structure. The settlements, however, do suggest that there may be some relative movement between the backfill and the building during a future earthquake. Accordingly, there may also be some relative movement between the pipelines leading into the building and the building structure. Because of the competency of the soils underlying the building, it is anticipated that soil/structure effects at the site would be relatively small. Estimation of the magnitude of deformation between the pipeline and the Control Works building is beyond the scope of this report. While not necessarily judged as being a high hazard, certain remedial techniques such as liners or flexible connections may be employed to improve the earthquake performance of the pipe at its interface with the building. From a geotechnical perspective, however, the relative competency of the building foundation soils and backfill soils would not warrant any additional studies of the pipeline and the pipe/building interface.

9.3 Structure Evaluations

This facility is comprised of three interacting structures: the concrete and masonry building structure and two surge tanks. These structures interact with one another through the reinforced concrete slabs at two levels. The degree of interaction is complicated by the existence of the 1/4-in. gap between the concrete slabs and the surge tanks.

In order to adequately capture the seismic behavior of the facility, several elastic models were developed. The initial bounding models assumed the gap to be fully open, and the building structure and the standpipes were analyzed as independent structures. This initial analysis resulted in the surge tanks being non-operational when subjected to the postulated Class I seismic loads because of overturning problems.

A second bounding analysis was performed which assumed that the gap was closed at all times (Figure 9.1-10).

The ANSYS model used for these analyses properly accounts for the three dimensional torsion effects which occur because of the lack of symmetry in the building structure's perimeter walls. Because of the lack of material and original design information, the following parameters were assumed:

f'_c = 3000 psi

f_y = 40 ksi for reinforcing steel

f_y = 28 ksi for steel tanks

f_y = 33 ksi for anchor bolts

and

Allowable soil bearing capacity = 12 ksi for dynamic loads

Beam elements were used to model each of the four exterior walls. The surge tanks were modeled by beam elements along their centerline. These were connected to the building at the roof and first floor levels with springs representing the in-plane shear stiffness of the slabs.

The lower part of the surge tanks is filled with concrete (Figure 9.1-6). Their stiffness at this level is much greater than that of the 66-in. diameter lines that connect to the surge tanks at their base. Therefore, the additional stiffness effect of the 66-in. diameter connecting pipes was ignored. The stiffness of the unreinforced masonry window infills was also ignored in the structural model. This assumption is conservative because the building period is in the ascending branch of the response spectra. Conservatively estimating a longer period for the building structure will result in the model being subjected to larger spectral accelerations.

Independent soil springs were modeled for the separate building and standpipe foundations. The contained fluid was modeled using the Housner method. This analysis demonstrated that the building structure can effectively contribute to the stability of the surge tanks if the concrete slabs maintain their structural integrity and the gap remains closed.

The effect of the 1/4-in gap was studied using an iterative approach to balance the deformation and stiffness of the different elements as follows:

a) The gap, when fully open, has a small stiffness because of friction and the asphalt material inside the gap.

b) The gap stiffness is infinite when the 1/4-in. space is closed.

c) The rocking stiffness decreases as the soil stress increases because of the soil bearing against the base of the surge tank foundations.

d) The building slab and shear wall elements decrease in stiffness due to concrete cracking as the gap closes and the building takes the load from the surge tanks.

The iterative approach proceeds as follows:

a) The stiffness of the different elements is estimated for the initial elastic model.

b) The resulting displacements are used to verify the state of stresses and stiffnesses previously estimated and corrections are made to repeat the cycle until all stiffnesses and resulting displacements agree.

The results of the evaluations are presented in Table 9.3-1.

The final iteration results, summarized in Table 9.3-1, indicate that the 1/4-in. gap between the roof slab and the tanks will eventually close and the slab will provide effective lateral support to the tanks when seismic loads reach high enough levels, as in the case of a Class I earthquake.

The most significant consequence of this lateral support is that soil bearing stresses, because of overturning, are not above allowable levels as they would be without the slabs supporting the surge tanks.

The floor slab and shear walls are stressed to levels above their cracking strength, but being reinforced they have a sufficient capacity margin and do not reach yield. The stiffness of the cracked slabs and shear walls is 10 to 20% of the uncracked sections.

Table 9.3-2 shows a comparison of design shears for the Control Works facility using different criteria.

Table 9.3-1

Control Works Facility (High Priority)
Final Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
1/4" gap between roof slab and tank	N/A	The gap will eventually close and the slab will provide effective lateral support to the tanks
East shearwall (shear)	0.92	(Concrete cracking
West shearwall (shear)	0.89	damage takes place in
Roof slab (shear)	0.75	the slabs and shearwalls
Floor slab (shear)	0.52	but there is no failure)
Building Foundation (soil bearing)	0.62	Adequate
Tank shell (compression buckling)	0.29	Adequate
Tank anchor bolts (tension)	0.06	Adequate
Tank foundation (bearing)	0.89	Adequate

Facility Status **Operable**

Table 9.3-2

Control Works Facility (High Priority)
Design Base Shear Comparison

1988 UBC	2,563 kips
Class I Earthquake	2,889 kips
Class II Earthquake	1,791 kips

9.4 Equipment and Contents Evaluations

The equipment in the Control Works Facility is primarily piping and valves. The majority of the piping is 3 to 6 ft. in diameter and is constructed of riveted steel.

The pipes are supported vertically at regular intervals. However, there is no specific lateral restraint system for the pipes. The longest horizontal span between supports is about eight pipe diameters, or 40+ ft. for the 66-in. bypass line. This length of horizontal span will not pose a problem for the steel pipe. It should also be noted that the system is redundant in many ways, and it is fully valved to allow the system to be operated in the event of an individual failure.

One area of potential concern is the pipe penetrations through the below grade walls. These large pipe penetrations are well below grade in what is assumed to be competent soil. The computed seismic displacements at this elevation are small. The construction documents do not detail the penetrations. It is assumed that the buried concrete pipes or tunnels transition to steel at the face of the concrete wall. The concrete pipe-to-wall interface is the one area most susceptible to damage. The calculated seismic displacement is small enough to not be a concern by itself. However should there be some settlement-induced stresses, these combined with a large earthquake might cause damage at this interface.

It is recommended that as part of regularly scheduled maintenance, these pipe/wall interfaces be carefully inspected. Should they be significantly cracked, a short liner or flexible connection should be installed.

9.5 Summary and Conclusions

9.5.1 Geotechnical

There does not appear to be any major geologic hazards that would threaten the earthquake performance of the Control Works building. Differential movements may occur between the buried pipeline and the Control Works building during the Class I event. While not considered to be a major hazard, various remedial techniques such as lining or flexible joints may be installed to improve earthquake performance of the connection between the pipe and the building.

9.5.2 Structures and Equipment

a) The evaluation of the Control Works structure and surge tanks, as described above, was performed using a water elevation in the surge tanks of 488 ft., as was indicated in the original design detail drawing shown in Figure 9.1-6. At the conclusion of the vulnerability study, it was determined that the actual water elevation is 493 ft. (5 ft. higher). This higher elevation will increase the stresses in the Control Works structure, which is

already stressed close to the allowable limit. Further study and analysis will be required to determine the impact of the increased water level on the final results. Table 9.5-1 summarizes the results of the seismic evaluations which used a surge tank water level of elevation 488 ft. The estimated cost of the reanalysis effort, which would consider the operational water level of 493 ft., is provided in Table 9.5-2.

b) In the event that the stresses in the Control Works structure exceed the allowable levels as a result of the reanalysis, it may be necessary to replace all of the unreinforced masonry infill in the windows with structural reinforced concrete infill in order to provide sufficient lateral load resisting capacity. A cost estimate for this potential seismic upgrade work is provided in Table 9.5-2.

c) The majority of the equipment in the Control Works Facility consists of miscellaneous large-diameter piping and valves. This equipment is well-supported and quite rugged, and should remain functional following a Class I seismic event.

Table 9.5-1

Summary of Preliminary Seismic Evaluations
Control Works Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	More analysis required
2	Piping and Valves	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 9.5-2

Summary of Upgrade Recommendations
Lake Youngs Control Works Facility
(Based on 1989 Dollars)

Facility	Vulnerability[1]	Facility Priority	Estimated Costs: Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Control Works	Moderate	High	\$30,000	\$32,800	\$24,200	\$90,000	\$177,000	\$7,300	\$184,300	±30%	Contractor

[1] Vulnerability refers to a combined ranking of both the structure and any associated equipment.

10.0 WATER OPERATIONS CONTROL CENTER

A seismic risk analysis has been performed on the Water Operations Control Center (WOCC). Each building at the site has been classified as to its vulnerability. A site visit was conducted where structural and architectural items were inspected. The existing drawings were also examined for potential seismic design flaws.

The majority of the complex was built in 1972 and is in relatively good condition. Buildings constructed at that time were designed for seismic code forces less than those specified for this project. However, the designers were aware of seismic load considerations and properly tied the buildings together in such a manner that they will perform adequately with minor seismic upgrades.

10.1 Facility Description

The Water Operations Control Center is comprised of a cluster of buildings. The main buildings in the group are the Administration Building, a two-story cast-in-place concrete structure and the Warehouse, Pipe, and Carpentry Shop. a tilt-up structure with a prestressed tee roof. Several pre-1972 structures have been renovated and are still in use at the site.

10.2 Geologic Hazard Assessment

The subsurface soils in the vicinity of the WOCC consist of alluvium and fill materials overlying glacially consolidated sediments. (Zone III on Figure 4.3-1). The surficial alluvial and fill materials are loose to medium dense, as evidenced by observed cracks in sidewalks, foundation walls, and separation of pavements and landscaping from buildings that are supported upon piling.

It is quite likely that the surficial alluvium and fill soils would liquefy during the Class I earthquake. Liquefaction of these soils would likely result in differential settlement of shallow spread footing foundations. These settlements may lead to major distress and structural damage of buildings. Buildings supported upon piling founded in the dense, underlying glacial sediments would be expected to perform relatively well. Although no specific evaluation was made of the freeway ramps which cross over the WOCC facility, disaster planning efforts should consider the potential effects of the collapse of these ramps since similar ramps have been shown to be vulnerable in past, major earthquakes.

10.3 Administration Building

10.3.1 Facility Description

The Administration Building is located on Airport Way South at the center of the Water Operations Control Center. The building is a two-story, cast-in-place concrete shear wall building with a pile foundation. The first and second floor plans are shown in Figure 10.3-1. The building, roughly rectangular in shape, has a North-South dimension of 120 ft. and an East-West dimension of 100 ft. The building is constructed around a central core, which on the ground floor measures 40 ft. by 40 ft. and is currently being used as a meeting/briefing area. At the second floor, the core dimensions are 60 ft. by 80 ft. and function as a lecture room and the control center. The first floor story height is 13 ft. with a second story height at the core of 16 ft., for a total building height of 29 ft.

The foundation system consists of columns supported by 25-ton treated timber piles and a 4-in. slab on ground. The second story and roof are constructed of concrete joists at roughly 3 ft. on center with a 3-in. slab. The second floor 14-in. joists, which compose the outer corridor, span 20 ft. from the exterior beam and column system to the central core shear walls. The second floor 19-in. core joists span 40 ft., wall-to-wall, and are depressed for a 2-1/2-in. non-structural light weight topping slab. The roof framing is similar to the second floor framing with 12-in. corridor joists and 19-in. core joists.

10.3.2 Structure Evaluations

The structural evaluation consisted of an ATC-14 type inspection of the building. This inspection is comprised of a structural review of the gravity and lateral systems and a nonstructural investigation focusing on life safety issues. The building was evaluated against two earthquakes: a Class I-Zone III earthquake with a 500 year return period, and a Class II-Zone III earthquake with a 100 year return period. A 5% damping ratio was assumed for the building.

10.3.3 Lateral and Gravity Capacities

The Seattle area has not experienced a major earthquake since the construction of the building was completed in 1972. Therefore, the only lateral loading the building has been subjected to is wind loading. This limited the investigation to a structural capacities vs. seismic demand check of items determined to be general areas of weakness in buildings of this type. To determine the demand on the structural elements, it is necessary to determine the basic response of the building to dynamic loads.

The estimated natural period of the building is 0.145 seconds in the E/W direction and 0.132 seconds in the N/S direction. The 0.145

second period was conservatively used in both directions for the analysis. This period resulted in the following accelerations of the center of mass and base shears for the building:

Type	Accel (g)	Shear (kips)
Class I	0.7	3,069
Class II	0.44	1,929

The Class I force is similar in magnitude to the force one would expect, if one was designing a structure to have an elastic response to a seismic occurrence.

The building is a concrete shear wall building. The lateral response in a building with concrete (rigid) diaphragms and concrete walls is dictated by the location of the center of mass of the building and the center of rigidity of the wall system. A perfect shear wall building has its walls arranged so that the center of mass and the center of rigidity align, minimizing global torsion. When, as in the office/meter shop building first floor, the center of mass and rigidity do not align, an additional torsional force is introduced. In addition, the wall layout on the ground floor is not reproduced on the second floor, causing the shear forces to transfer out of the upper story walls and travel through the second story diaphragm to the lower shear walls. This irregularity increases the diaphragm loads and can cause shear and tension problems in the columns supporting the upper walls.

It was necessary to develop acceptance criteria for the two levels of earthquakes that the building was evaluated against. The criteria used for allowable stresses was as follows: Class I, the unfactored stress divided by the yield capacity shall result in a ductility demand that is reasonable considering the type of failure mechanism involved; Class II, if Class I requires an unreasonable ductility demand determine the ductility requirement for a Class II earthquake.

The critical element which governed the response of the building was the cast-in-place concrete walls. For the Class I earthquake, the walls were checked against three types of failure; shear, flexure and foundation failure. The shear failure mode has a ductility demand of 2. In flexure, the largest ductility demand of the major walls was 4.

The final category checked was the stability of the walls and foundations. For this calculation the entire tributary structure weight was used. Cross walls were also used where applicable. The

largest overturning-to-dead load moment ratio was 3. For all the major walls, the walls will yield in flexure first. The reinforcing steel is detailed with proper tension development lengths and the ends of the walls are tied as columns. The walls span between major grid lines where column pilecaps exists. These pile caps are not capable of transmitting tension to the piles. These ductility demands for shear, flexure and foundation overturning are reasonable and therefore the Class II earthquake was not investigated.

10.3.4 Equipment and Contents Evaluations

The main areas of concern pertained to how non-structural items were attached to the building. The site visit identified several items which required seismic upgrading, as listed below.

a) In the boiler room, neither the boiler nor the vertical tank were secured to the floor with anchor bolts. It is possible that during a seismic event these large tanks could shift and rupture their attached lines. It is recommended that anchor bolts be added.

b) In the meter shop, the 9-ft. tall wood shelves did not appear to be secured to the floor. A 10-ft. tall filing cabinet was also not secured. While these items do not in and of themselves constitute a life threatening situation, they may block timely access to the room. In this same room, the metal shelves were properly braced and secured. It is recommended that the wood shelves be properly braced and secured.

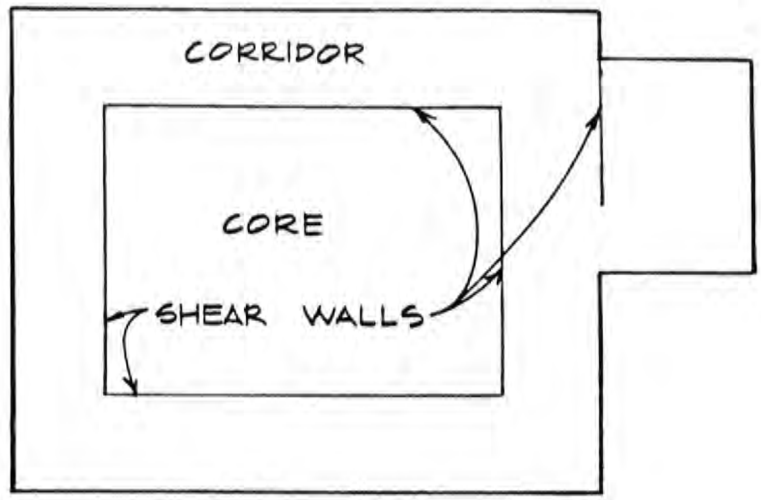
c) On the second floor of the building, the metal control panels in the control center are not secured. It is essential that these panels be modified and adequately braced as their ability to function is essential.

d) The free-standing precast plant containers on the second floor need to be removed or attached to the structure.

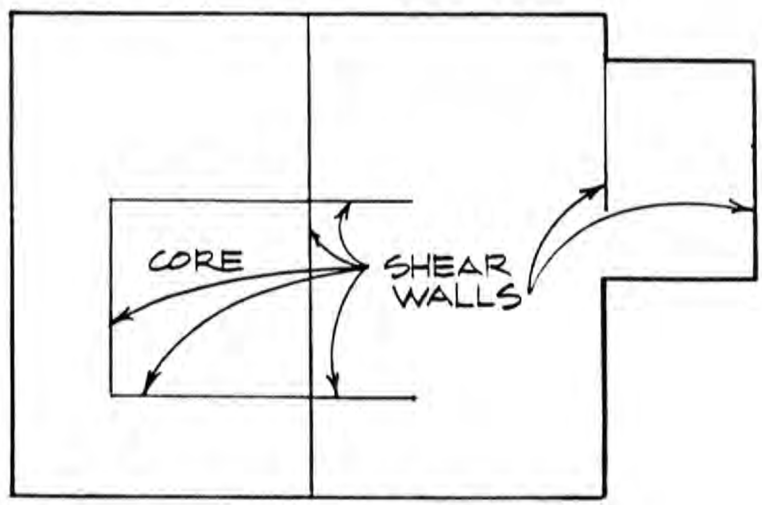
10.3.5 Summary and Conclusions

a) The Administration Building structural system is adequately designed to respond to the design earthquakes.

b) The interior contents of the building with the above modifications should adequately respond to the design earthquakes.



SECOND FLOOR/ROOF
1" = 30'



FIRST FLOOR
1" = 30'

Administration Building

Figure 10.3-1

10.4 Warehouse

10.4.1 Facility Description

The Warehouse Building was constructed in 1972 and is located adjacent to the Administration Building on the south side. The building, shown in Figure 10.4-1, is rectangular with an East-West dimension of 100 ft. and a North-South dimension of 300 ft. The roof height is 22 ft. 6 in. The building foundation system is composed of a pile system supporting the columns and structural slab. The roof is constructed of prestressed single tees which span 100 ft. The flange portion of the tee forms the roof deck with no topping slab. The tees are supported on precast perimeter beams. The tilt up precast panels which form the exterior cladding of the building and function as shear walls tie into the perimeter beams and the structural slab.

The interior of the building has a mezzanine at the north end and a series of offices and work areas along the east side. These interior partitions are wood bearing wall structures.

10.4.2 Structure Evaluations

The structural evaluation consisted of an ATC-14 type inspection of the building. This inspection is comprised of a structural review of the gravity and lateral systems and a nonstructural investigation focusing on life safety issues. The building was evaluated against two earthquakes: a Class I-Zone III earthquake with a 500 year return period, and a Class II-Zone III earthquake with a 100 year return period. A 5% damping ratio was assumed for the building.

10.4.3 Lateral and Gravity Capacities

As with the Administration Building, the Warehouse Building structure has not been subjected to any seismic events and thus must be evaluated on its capacity/demand ratios in areas of potential weakness for buildings of this type. To determine the demand on the structural elements, it is necessary to determine the basic response of the building to dynamic loads.

The estimated period of the building is 0.113 seconds in the E/W direction and 0.06 seconds in the N/S direction. These periods result in the following accelerations of the center of mass and base shears for the building:

	East/West		North/South	
Type	**Accel (g)**	**Shear (kips)**	**Accel (g)**	**Shear (kips)**
Class I	0.65	2,429	0.45	1,543
Class II	0.41	1,532	0.28	963

A potentially weak area in a building of this type is the ductility of the discrete connectors. For this building, the diaphragm connections at the roof and shear panel connections were investigated.

The roof diaphragm connectors are composed of bent reinforcing steel bars protruding from the tee flanges at 6 ft. o.c. An additional bar is laid parallel to the protruding bars and welded to form the connection. At the point of maximum shear in the diaphragm, there is a demand/capacity ratio on these connectors of 3.84 for the Class I earthquake. This ratio is reasonable as the welds are stressed longitudinally and the failure mechanism is a shear friction tensile failure of the reinforcing steel. However, the transposition of the detail from the design drawings to the field may have caused a potential problem. As detailed, the welded connection is on straight rebar but with field fit up problems it is likely that welding occurred on areas where the rod was bent. This combination of bending and welding often causes embrittlement problems which reduce the ductility of the connector to zero. These connectors should be visually inspected and replaced where welded on bends.

The precast wall panels, 18.5 ft. high by 10 ft. wide, are connected to the gravity structure at six locations, two connectors at the panel top to the spandrel beam, one each side to either adjacent panel or column, and two at the base of the panel to the structural slab. See Figure 10.4-2 for the south wall layout. The top and side connectors are composed of embedded threaded inserts, bolts and plates. The bottom connectors are bent reinforcing steel which is welded to a mating piece protruding from the slab. The panels appear to be designed to act independently, placing the bottom connector in both shear and tension. The weld is then stressed both longitudinally and transversely. The base connectors are the weakest link in that the failure mechanism is the weld being stressed in tension and in shear. This problem is magnified by the possible embrittlement of steel caused by welding on areas where the reinforcing steel is bent.

The demand/capacity ratio of the base panel connectors with the panels acting independently is 18 for the Class I event. This high ratio occurs at the short north/south walls and is unacceptable. The panel layout as seen in the sketch offers the opportunity for the panels to be tied together so that the panels transmit the shear and

the columns serve as the overturning members. While this approach, which requires grouting the top connectors to transmit bearing to the beams and adding additional side connectors, reduces the capacity demand ratio to 13, it is still not an adequate solution.

The ability of the north and south walls to resist the loads is limited by the roof framing system which transfers the dead load to the east and west walls and does not allow for any tension to be transmitted to the piles. The most economical solution is to add two, 40-ft. long interior shear walls running east/west and drag struts to connect these walls to the roof. These walls, as shown in the Warehouse plan, are schematically sized at 14-in. thick and transfer the overturning loads to auger cast piles capable of taking uplift. The current layout of the Warehouse is along east/west lines and these partial walls, while reducing visibility, will not greatly affect operations. The new walls running east to west increase the capacity of the east/west lateral system so that the demand/capacity ratio of the system is 2.

The long east and west walls resist a north/south occurring earthquake. As currently designed, the demand/capacity ratio for the panel base connectors is 3. This assumes that the panels act independently and that the welds fail due to tension/shear. This ratio can be reduced to 1 by connecting the panels so that they act as a shear panel and transfer the overturning to the columns. This can be accomplished by grouting the upper panel connectors, adding additional side panel connectors and repairing and replacing base connectors welded at bends.

At this time, the Warehouse building constitutes a high risk in the event of a significant Class I-type seismic event. The east/west lateral system needs to be upgraded by adding shear walls. The north/south lateral system composed of the east and west walls has sufficient strength for a low-level earthquake, however the performance of these walls can be significantly improved. All the welded precast connections need to be inspected and replaced as required.

10.4.4 Equipment and Contents Evaluations

During the site visit, the following items were observed:

a) The large storage racks were securely tied to the building for loads out of plane. However, in the longitudinal plane, the racks were able to be moved or excited by pushing against them. It is recommended that bracing be added.

b) Large heavy items are currently being stored on the upper shelves of the racks. These items need to be moved to a low shelf or restraining devices need to be added to the front of the racks.

c) Chlorine tanks are stored at the south end of the building. The large tanks stored horizontally are adequately secured but it appeared that a more secure method should be devised for the smaller vertical tanks.

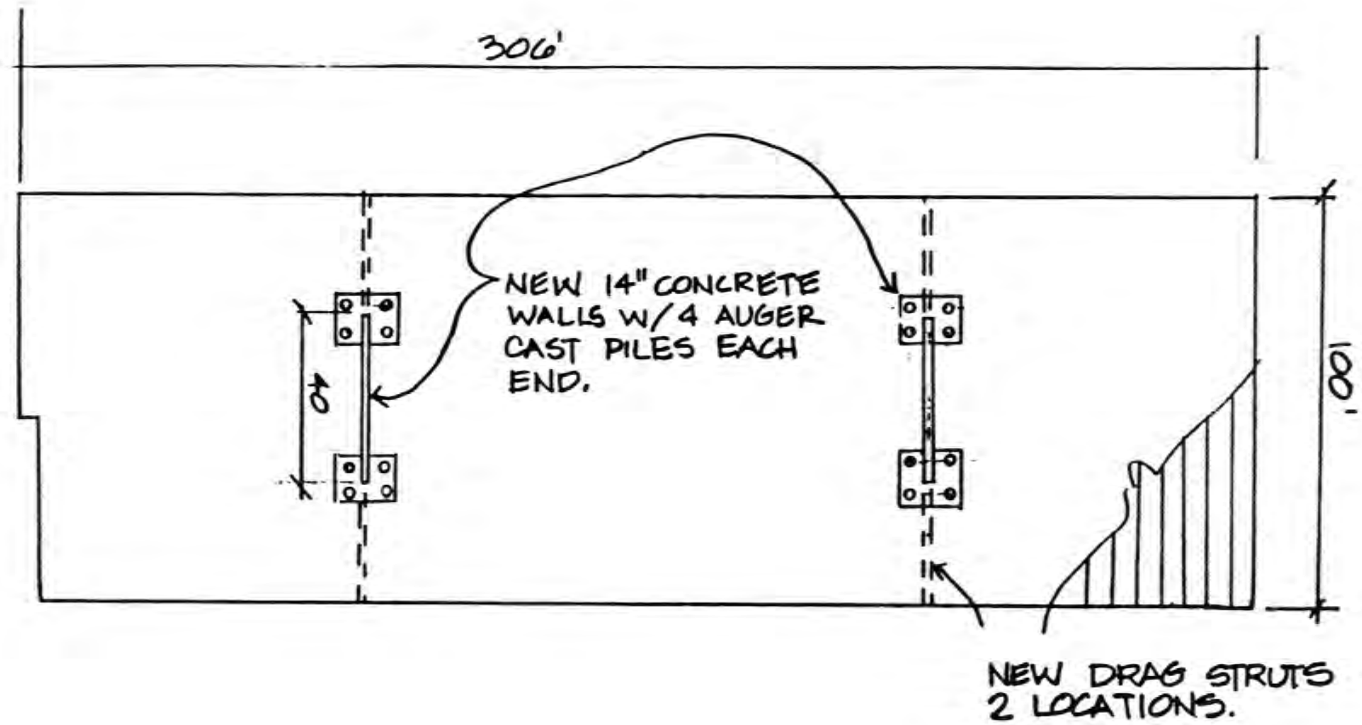
10.4.5 Summary and Conclusions

a) At this time, the Warehouse Building constitutes a risk in the case of a seismic event. The east/west lateral load resisting system, comprised of the north and south walls, does not have sufficient strength or ductility to adequately resist the design earthquakes.

b) The north/south lateral load resisting system, comprised of the east/west walls, has sufficient strength to withstand a Class I-level earthquake; however the performance of those walls can be significantly improved at a minor cost as described above.

c) All precast connections in the roof and the walls which involve welding of bent reinforcing need to be inspected and replaced where necessary. The wall panel connections at the base are visible from the interior of the building once the grout pocket is removed. The roof connectors can be exposed by removing the roofing material. The connectors are not visible from the interior of the building.

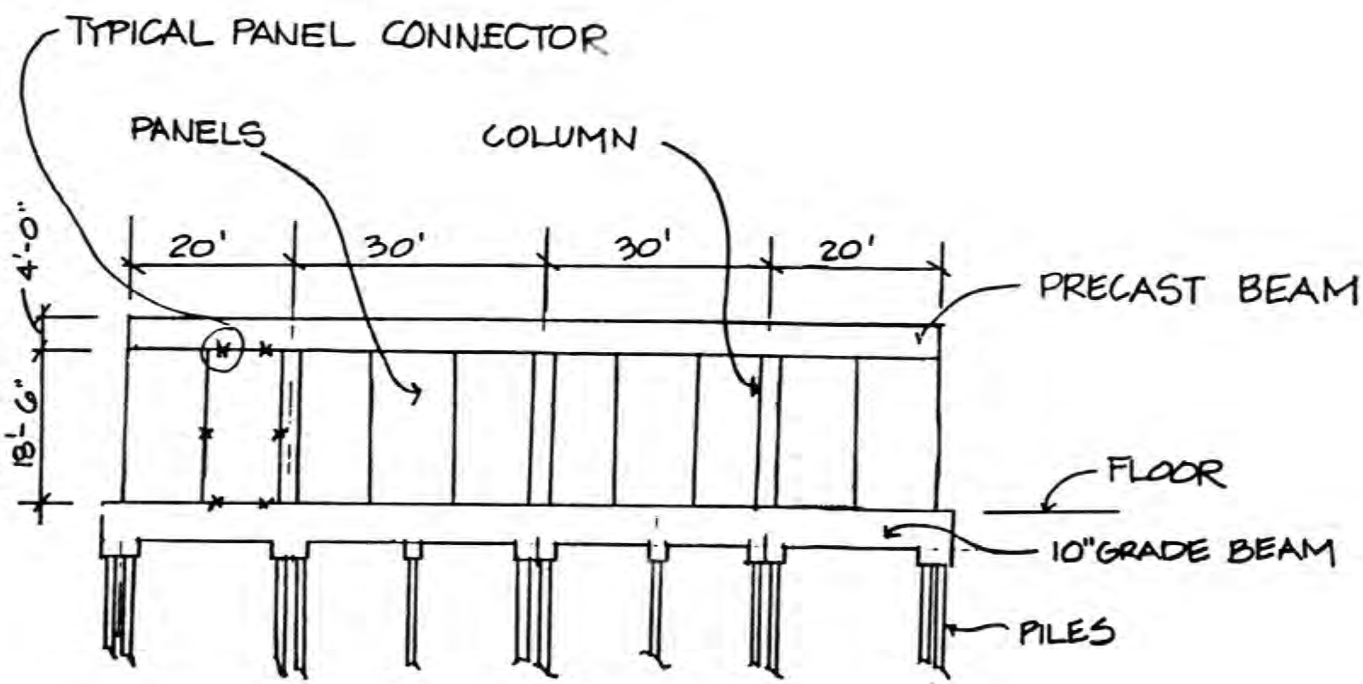
d) The interior contents of the building, with the above modifications described for the storage racks, should adequately resist the design earthquakes. The 150-lb. chlorine cylinders located near the outside south wall of the Warehouse Building should be more properly secured.



Warehouse Plan

Figure 10.4-1

2048 00



Warehouse - South Elevation

Figure 10.4-2

10.5 Flammable Liquid Storage Building

10.5.1 Facility Description

This building is located due east of the Warehouse and is used for the storage of flammable materials. The age of this building is not known. The north wall of the building is composed of unreinforced hollow clay tile. The west wall appears to be unreinforced masonry. The structure is severely cracked, and the consultant did not enter the building to observe the east wall construction or the lateral system.

10.5.2 Structural Evaluation

The building is in poor condition and in the event of a design earthquake the walls would probably collapse.

10.5.3 Summary and Conclusions

The continued use of this building should be carefully considered. While the building is not required to be operational after an earthquake, its location in the yard and the ensuing pile of rubble may interfere with other operations. The contents of this building will become damaged and possibly ignited if the building walls collapse. It is therefore recommended that this structure be demolished and replaced with a suitable storage facility. The cost estimate in Table 10.8-2 includes an amount for the demolition of the existing structure and the construction of a new one. It is envisioned that this new structure be of concrete masonry block construction in order to provide for fire protection between adjacent portions of the building.

10.6 Pipe/Carpentry Shop

10.6.1 Facility Description

This building is located in the North-East corner of the yard. The foundation system is slab-on-ground and the columns are supported on timber piles. The roof is composed of precast, prestressed double tees. The tilt up precast wall panels form the exterior cladding and function as shear walls. In the Pipe Shop, a mezzanine exists which is constructed of 2 x 12-in. members supported on an 8-in. c.m.u. bearing wall.

10.6.2 Structure Evaluations

The structural evaluation was performed using an ATC-14 type of inspection. This inspection is comprised of a structural review of the gravity and lateral systems and a nonstructural investigation focusing on life safety issues. The building was evaluated against two earthquakes: a Class I-Zone III earthquake with a 500 year return period, and a Class II-Zone III earthquake with a 100 year return period. A 5% damping ratio was assumed for the building.

10.6.3 Lateral and Gravity Capacities

The building, constructed in 1972, has not been subjected to any seismic events and thus must be evaluated on its demand/capacity ratios in areas of potential weakness for buildings of this type. To determine the demand on the structural elements, it is necessary to determine the basic response of the building to dynamic loads.

The estimated period of the building is 0.1 second in the E/W and 0.12 seconds in the N/S. The 0.12 second period was used for the analysis in both directions. This period resulted in the following accelerations of the center of mass and base shears for the building:

Type	Accel (g)	Shear (kips)
Class I	0.65	905
Class II	0.41	565

A potential weakness in a building of this type is the ductility of the discrete connectors. For this building, the diaphragm connection and the roof and shear panels connections were investigated.

The diaphragm connector is composed of embedded angles with a plate welded to the two pieces. The weld is stressed longitudinally.

The probable failure would be a shear friction failure with the angle breaking from the flange of the double tee. The demand/capacity ratio of these connectors is 1.5 for a Class I event.

The precast wall panels, 18.5 ft. high by 10 ft. wide, are connected to the gravity structure at eleven locations, three connectors at the panel top to the spandrel beam, three each side to either adjacent panel or column, and two at the base of the panel to the structural slab. The top and side connectors are composed of bent reinforcing steel which is welded to a mating piece protruding from the beam, column or panel. The bottom connectors are embedded steel plates. The panels appear to be designed to act independently by placing the bottom connector in both shear and tension. The weld is then stressed both longitudinally and transversely. The panel connectors are designed to have the same ultimate capacity with a demand/capacity ratio of 1. The top and side connectors have a potential problem in possible embrittlement of steel since the bent rebar was welded. All the connections where this potential problem exists should be inspected and repaired, as required.

The building layout with the double tees running east/west supply no dead load to the north/south walls. These walls, which comprise the east/west lateral system, have an overturning problem. As currently designed, the pile caps are not capable of transmitting this overturning load to the timber piles. This problem needs to be resolved so that the system can work as a unit. One possible solution is to cement a prestress rod to the pile and pile cap assuming that the piles are capable of taking uplift. If the full capacity of the panels can be developed, the demand/capacity ratio for the north/south walls is 1.

10.6.4 Equipment and Contents Evaluations

The Pipe/Carpentry Shop floor houses numerous machines. All machines should be inspected by SWD maintenance to verify that the machines were properly installed. The installation should have included anchoring the machine legs to the slab with galvanized expansion anchors. Where these anchors were not originally installed, or have been removed, they should be replaced.

10.6.5 Summary and Conclusions

The Pipe/Carpentry Shop Building will adequately withstand the design earthquakes once the proper connection of the building to the foundation is accomplished and the panel connections at the top and sides of the precast panels are investigated and strengthened, as necessary. When all large pieces of equipment are secured to the slab, the contents of the building should also perform adequately.

10.7 Vehicle Maintenance/Storage Building

10.7.1 Facility Description

This building is located at the north end of the yard. The structure was renovated in 1972, however, the original age of the building is unknown. The floor is a new slab on ground and the columns are supported on spread footings. In the vehicle maintenance shop, new footings were added to support a bridge crane. The roof system is comprised of wood joists supported by glu-lam beams. The north, south and east walls are c.m.u. from the original building. The west and center walls are new c.m.u. walls.

10.7.2 Structural Evaluation

The building appears to be in poor condition and at this time the original drawings from the pre-1972 building were not available.

The following items were observed during the site visit:

a) The south wall of the building is severely cracked both along the joints and through the c.m.u. blocks. The east portion of the south wall is displaced 1 in. horizontally, 3 ft. above the ground. The outside bearing edges of the c.m.u. blocks are not in contact and the load is carried through bearing on the interior web of the blocks. From this type of displacement, it appears that the existing walls are not reinforced.

b) The interior racks in the east portion of the building were not secured to either the roof or the floor. Large items stored on the upper shelves could be hazardous. The racks along the south wall were attached to the wall.

10.7.3 Summary and Conclusions

a) The south wall currently constitutes a hazard. This wall should be replaced and the remaining older walls, if unreinforced, should also be replaced as they constitute a hazard during a seismic event.

b) The racks in the warehouse area should be properly secured to the floor. The racks along the south wall should be attached to a sound wall.

c) This building is considered essential to be operable immediately following an earthquake. The south wall, because of its current condition and height, constitutes a life safety hazard in a moderate earthquake.

10.8 Summary and Conclusions

10.8.1 Geotechnical

The surficial alluvial and fill soils underlying the Water Operations Control Center may potentially liquefy during a Class I earthquake. Liquefaction of these soils would be detrimental to buildings supported upon shallow spread footing foundations. Buildings supported upon pilings extending to the underlying, competent, glacially consolidated sediments, should perform relatively well. Liquefaction of the surficial sediments may also be disruptive to utility lines leading into the buildings. Finally, the potential of collapse of the elevated freeway ramps which pass over the facility should be considered in the Emergency Response Plan.

10.8.2 Structures and Equipment

The Water Operations Control Center is composed of several types of buildings with various abilities to respond to the design earthquakes. The Administration Building will adequately respond to the earthquakes. The contents of this building, specifically the control panels on the second floor, need to be attached to the building. The Warehouse Building does not have sufficient strength or ductility to respond to the design earthquakes. The storage racks of this building need restraining devices. The Paint Storage Shed is an unreinforced masonry building which should be replaced. The Vehicle Maintenance/Storage Building shows signs of severe cracking and should be repaired. The Pipe/Carpentry Shop requires foundation modifications and the anchorage of the machinery in this building needs to be verified/repaired.

A summary of the seismic performance of the Water Operations Control Center and the associated cost for seismic upgrades is provided in Tables 10.8-1 and 10.8-2.

Table 10.8-1

Summary of Preliminary Seismic Evaluation
Water Operations Control Center

Item	Description	Facility Priority	Class I	Class II
1	Administration Building	High		
	Structure		1	1
	Components		2	1
2	Warehouse	High		
	Structure		3	2
	Components		2	1
3	Storage Shed	Low		
	Structure		3	3
4	Pipe/Carpentry Shop	High		
	Structure		2	1
	Components		1	1
5	Vehicle Maintenance/ Storage Building	High		
	Structure		3	3
	Components		2	1

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operability)

Table 10.8-2

Summary of Upgrade Recommendations for Water Operations Control Center (Based on 1989 Dollars)

			Estimated Costs								
Facility	Vulnerability[1]	Facility Priority	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Office Building	Moderate	High	$ -0-	$ 4,000	$ 2,000	$ 12,500	$ 18,500	$ 1,000	$ 19,500	±30%	SWD
Warehouse	High	High	2,000	61,500	49,100	235,000	347,600	19,000	366,600	±30%	Contractor
Storage Shed	High	Low	-0-	16,500	16,500	60,000	93,000	4,900	97,900	±30%	Contractor
Pipe/Carpentry Shop	High	High	-0-	16,500	10,300	81,000	107,800	6,600	114,400	±30%	Contractor
Vehicle Maintenance	High	High	-0-	34,500	23,300	143,000	200,800	11,600	212,400	±30%	Contractor
Totals:			$2,000	$133,000	$101,200	$531,500	$767,700	$43,100	$810,800		

(1) Vulnerability refers to a combined ranking of both the structure and any associated equipment.

11.0 CHLORINATION FACILITIES AND PUMP STATIONS

11.1 Facility Overview

The Seattle Water Department maintains pump stations throughout its service area to provide pressure levels above that which can be delivered by the Cedar River and Tolt pipelines. Higher elevation water pressure zones throughout the area are serviced by pump stations. The SWD facilities in the Seattle urban area, including those described in this section, are shown in Figure 11.1-1.

The pressure zones in the SWD direct service area, as shown in Figure 11.1-2, are assigned a number designation which reflects the static pressure head of the zone. Typically, each zone is pressurized by several pump stations or turbines, with at least one water tank or standpipe used for storage. Thus, there is generally a system redundancy, so that one pump station can be shut down without necessitating a significant loss of pressure in the zone.

One exception is the small 540 zone served by the South Augusta Street pump station. If the Augusta pump station is shut down, pressure in this zone reverts to Cedar River pipeline head.

A second exception is the 650 zone in a residential area of north Seattle served by the Dayton Avenue pump station. If the Dayton pump station is shut down this zone reverts to a 580 zone.

There are also a multitude of piping interconnections between zones in the system, with pressure regulated by pressure relief valves or orifices, which provide additional system flexibility. Thus, a given pump station which normally serves one zone, may also provide water to another zone if the necessity arises. Some pump stations are only utilized during high demand summer periods. Table 11.1-1 lists those zones pressurized by pumps or turbines.

Chlorination facilities are provided at the outlet of each open air reservoir in the system, and at the Boulevard and Riverton Heights well fields. Since this includes the chlorination that is performed at the Tolt, Landsburg and Lake Youngs facilities, the potable water supply is typically chlorinated two or three times between source and tap. Nonetheless, as residual chlorine levels drop with open air exposure, each chlorine facility is important to assure sanitary water.

Each facility is connected by telephone line telemetry to the SWD Control Center on Airport Way South in Seattle. The system is monitored and controlled through the telephone line telemetry. An important consideration in assessing the seismic vulnerability of a facility is the possibility of disruption of telemetry resulting from damage at the facility.

Table 11.1-1

PUMP STATIONS AND STORAGE FACILITIES BY PRESSURE ZONE

<u>575 Zone</u>

Storage: Beverly Park and S.W. Myrtle #1 and #2 Tanks
Pump Stations:

1) S.W. Trenton (Section 11.34)
2) Highland Park (11.17)
3) Burien (11.9)

<u>488 Zone</u>

Storage: S.W. Myrtle Reservoir, S.W. Myrtle #1 and #2 Tanks, Charlestown Standpipe
Pump Stations:

1) West Seattle (11.31)
2) S.W. Spokane (11.32)

<u>540 Zone</u>

Storage: Lake Forest Reservoir
Pump Stations:

1) S. Augusta Street (11.3)

<u>520 Capitol Hill - Queen Anne Hill Zone</u>

Storage: Volunteer and Queen Anne #1 and #2 Standpipes
Pump Stations:

1) Lincoln Park (11.21)
2) Broadway (11.8)
3) Volunteer Park (11.37)
4) Warren Avenue N. (11.39)

<u>520 First Hill Zone</u>

Storage: Volunteer and Queen Anne #1 and #2 Standpipes
Pump Stations:

1) First Hill (11.13)
2) Broadway (11.8)

<u>470 Zone</u>

Storage: Queen Anne #1 and #2 Standpipes
Pump Stations:

1) Interbay (11.18)

<u>580 Zone</u>

Storage: Richmond Highlands #1 and #2 Tanks, Foy Standpipe
Pump Stations:

1) North City (11.27)
2) Foy (11.14)
3) Bitter Lake (11.5)

Table 11.1-1 (Cont'd)

520 Maple Leaf Zone
- Storage: Maple Leaf Tank
- Pump Stations:
  1) Maple Leaf

650 Zone
- Storage: Dayton (11.10)
- Pump Stations:
  1) Dayton Avenue (11.10)

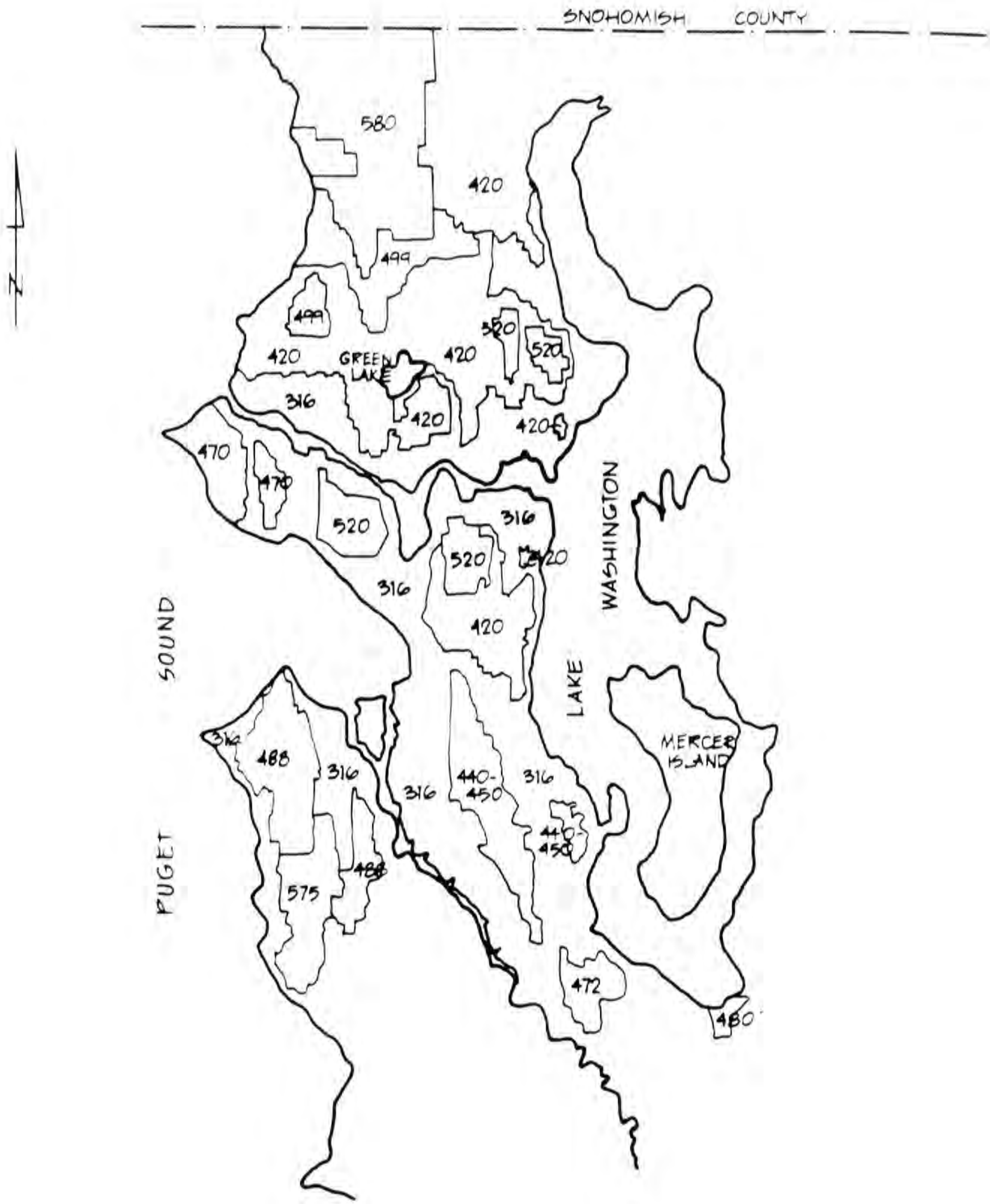


Figure 11.1-1
Service Area Pressure Zones

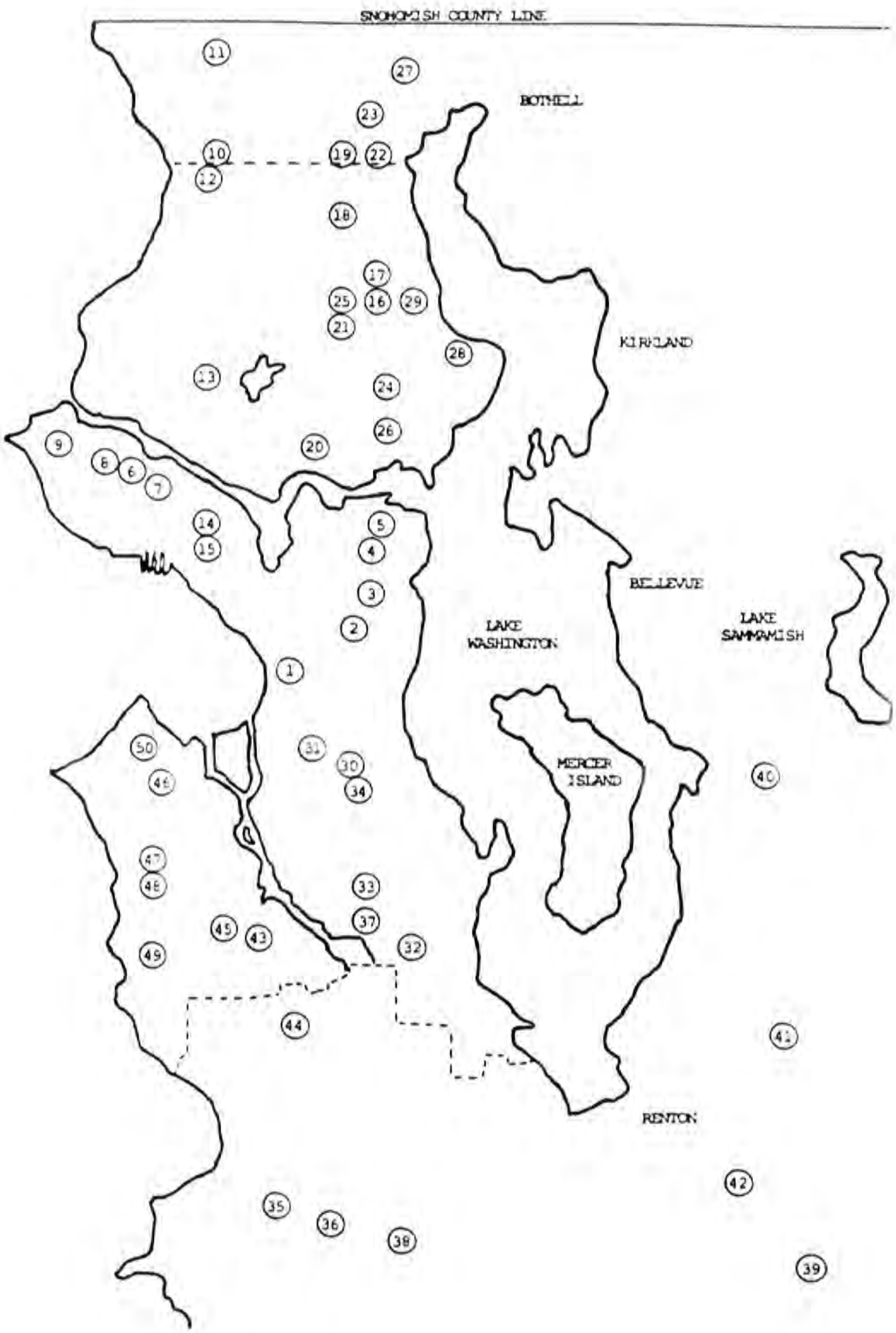


Figure 11.1-2
SWD Facility Locations (obtained from SWD)

CODE	ADDRESS	OFFICE OR SHOP	CHLORINE BUILDING	PUMPING STATION	RESV.	TANK OR STANDPIPE	CONTROL VALVE
1	821 2nd Ave. (Admin.)	1					
2	Nagle Pl. & E. Olive St.		1	2	1		4
3	12th Ave. E. & E. Jefferson			1			1
4	12th Ave. E. & E. Prospect		1	1	1	1	1
5	22nd Ave. E. & E. Boyer Ave.						1
6	23rd Ave. W. & W. Dravus St.			1			1
7	26th Ave. W. & W. Crockett St.						1
8	27th Ave. W. & W. Bertona St.		1		1		
9	38th Ave. W. & W. Prosper St.					1	
10	Dayton Ave. N. & N. 145th St.			1		1	
11	Fremont Ave. N. & N. 195th St.					2	
12	Linden Ave. N. & N. 143rd St.		1	1	1		2
13	Phinney Ave. N. & N. 53rd St.					1	1
14	Warren Ave. N. & N. Lee St.					2	
15	Warren Ave. N. & N. Valley St.			1			
16	Roosevelt Way NE & NE 82nd St.			1			2
17	Roosevelt Way NE & NE 86th St.					1	
18	4th Ave. NE & NE 115th St.			1 M			1
19	5th Ave. NE & NE 145th St.			1			
20	7th Ave. NE & NE 42nd St.						1
21	8th Ave. NE & NE 80th St.						2
22	8th Ave. NE & NE 145th St.						1
23	8th Ave. NE & NE 185th St.			1			1
24	12th Ave. NE & NE 73rd St.		1	1 M	1		
25	12th Ave. NE & NE 82nd St.		1	1	1		1
26	25th Ave. NE & NE 45th St.						1
27	45th Ave. NE & NE 195th St.		1		1		3
28	55th Ave. NE & NE 75th St.			1	1		2
29	82nd NE & NE Lake City Way			1			
30	1509 S. Spokane St. (Lab.)	1	1				
31	2700 Airport Way S. (WOCC)	1					
32	Beacon Ave. S. & S. Augusta St.			1			
33	Beacon Ave. S. & S. Kenyon St.						1
34	Beacon Ave. S. & S. Spokane St.				1		2
35	8th Ave. S. & S. 146th St.			1			
36	24th Ave. S. & S. 148th St.				1		2
37	51st S. & S. Leo St.						1
38	160th St. S. & Military Road			1			
39	18015 SE Lake Youngs Road	1	1	2	1		
40	SE 20th Ave. & 140th Pl. SE			1			2
41	SE 139th & 132nd Ave. SE			1			
42	SE 171st P. & SE 140th St.			3			
43	4th Ave. SW & SW Trenton St.		1	1 M		2	
44	4th Ave. SW & SW 111th St.					1	
45	8th Ave. SW & SW Henderson St.		1	2	1		2
46	33rd Ave. SW & SW Spokane St.			1			
47	35th Ave. SW & SW Morgan St.						1
48	35th Ave. SW & SW Myrtle		1		1	2	
49	38th Ave SW & SW Barton St					1	
50	39th Ave. SW & SW Charlestown					1	

Figure 11.1-2 (Cont'd)
SWD Facility Locations

11.2 Geologic Hazard Assessment

Except for the pump stations located at Interbay, Magnolia Bluff, and S.W. Spokane Street, the chlorination facilities and pump stations within the Seattle Water Department system are all located upon competent, glacially consolidated sediments (Zone II on Figure 4.3-1). These sediments may range in composition from glacial till to outwash sands and gravels. The combination of competent soil conditions and generally moderate topographic relief would typically preclude concern for the major earthquake-induced landslides or liquefaction.

The pumping stations at Interbay, Magnolia Bluff, and S.W. Spokane Street are located in areas deemed as having a high landslide potential (Zone III on Figure 4.3-1). The Interbay and Magnolia Bluff stations were classified as having a high landslide potential based upon inferred geological conditions consisting of sands overlying clays of the Lawton formation. These conditions have typically resulted in landslides at areas where groundwater flowing along the contact of the two materials emerges at the face of the slope. However, the slopes observed adjacent to the Interbay and Magnolia Bluff pumping stations have performed satisfactorily to date and do not show any major signs of distress or instability. Therefore, remedial action is not recommended to improve slope stability for these sites.

The hillside slopes adjacent to the S.W. Spokane Street pumping station, however, currently show signs of surficial creep or hillside instability. This instability is primarily occurring on the slope on the west side of the pumping station. The City of Seattle has installed a low (3-ft.-high) slab and rail retaining wall on the west side of 33rd Avenue to retain the hillside above the street. Nevertheless, soil creep has continued to occur over the top of the wall. It is not expected that this slope movement would constitute a major threat to the pump station; however, this slope movement may aggravate line breaks or leaks in the area. As this observed movement would not constitute a major threat to the operation of the pumping station, remedial action is not recommended. Subsurface cutoff drains could be installed on the water department property to locally improve the stability of the slope.

Although it appears that the chlorination facilities and pumping stations are generally located upon competent foundation bearing strata, the site geological reconnaissances noted the existence of cracks in the sidewalks and paved areas adjacent to many of these facilities. It would appear that these cracks are a result of settlement of the backfill placed adjacent to the buildings. These cracks are largely a maintenance nuisance and they do not necessarily represent a major earthquake hazard for the building. However, the existence of these cracks would suggest that the backfill adjacent to most of these buildings is loose and that there may be some soil/structure interaction between the building and the adjacent backfill. This may result in some relative movement between pipelines that enter the buildings and the walls of the building. This amount of relative displacement is anticipated to be small, although not specifically computed as part of this study.

In summary, the chlorination facilities and pumping stations are located upon competent, glacially consolidated sediments that are at minimal risk from liquefaction during the Class I earthquake. Additionally, these facilities have a relatively low risk of being affected by an earthquake-induced landslide. Only the S.W. Spokane Street pumping station is located in an active landslide area, which shows signs of surficial creep. It is unlikely that any earthquake-induced movement of the hillside in this area would constitute a major threat to the operation of the pump station. Continued creep in the hillside may be detrimental to pipelines leading into the pumping station. As this facility has performed relatively satisfactorily for the past 30 years, any remedial action to improve hillside stability, such as installation of subsurface drainage, would have a relatively low priority for installation.

11.3 S. Augusta St. Gate House and Pump Station

11.3.1 Facility Description

The S. Augusta St. facility, located near Beacon Ave. S. and S. Augusta St., was constructed in 1930 and supplies water to the 540 zone as well as the Leo St. area. The structure is cast-in-place reinforced concrete on continuous footings. The building measures 29 ft. by 14 ft. The concrete slab roof is supported by two longitudinal concrete walls and is 9 ft. above the interior slab on grade. The original window openings have been filled in with c.m.u. block.

The 24-in. gate valve, which can be manually or electrically operated, is normally open on the Cedar River pipeline #3 in order to feed water to the West Seattle pipeline. The 36-in. gate valve on Cedar River pipeline #1 and the 36-in. butterfly valve on Cedar River pipeline #2, both manually operated, are normally left closed. All these valves can only be independently operated on site.

11.3.2 Structure Evaluations

From the original construction documents the 8-in. exterior concrete perimeter walls are generally reinforced with #4 bars at 18-in. o.c. both ways each face, which is adequate reinforcing by today's current ACI code. A quick analysis of this structure concludes that some visible cracking or yielding may occur but shouldn't affect the structural integrity of the building. The fact that the openings have been filled in with block increases the lateral stability of the entire structure. Thus, we judge this structure as having a low vulnerability to seismic damage in a Class I-type earthquake.

11.3.3 Equipment and Contents Evaluations

Pumps

Four pumps are contained within the pump station. These pumps rest on concrete pedestals and are anchored with anchor bolts. This equipment is considered to have a low vulnerability to earthquake damage.

Electrical Control Panels

There are several electric panels that are located within the building. The electrical equipment is enclosed in rigid sheet metal cabinets. Due to the small width of the southern motor control panel, we recommend that the panel be anchored to the floor slab with four (4) 1/2-in. diameter kwik bolts with 4-in. embedment to prevent overturning during an earthquake.

Valves

The 24-in. and 36-in. gate valves and the 36-in. butterfly valve, along with their associated piping and controls, are all well supported and in good condition and have low vulnerability to seismic damage.

11.3.4 Summary and Conclusions

a) From the preliminary investigation of the original construction documents, it was not possible to obtain adequate information on some important details. However, when the ATC-14 checklist for concrete structures was evaluated, this building rated adequately.

b) The southern electrical control panel unit should be anchored to the floor for stability.

c) Table 11.3-1 summarizes the results of the seismic evaluation of the Augusta Pump Station. An estimate of remedial upgrade costs is presented in Table 11.3-2.

Table 11.3-1

Summary of Preliminary Seismic Evaluations
S. Augusta St. Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps/Valves	1	1	Well anchored
3	Electric control panel	3	2	Anchor motor control unit

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.3-2

Cost Estimate

Location: S. Augusta St. Pump Station

Date Constructed: 1930

Priority: High

Seismic Strengthening Objectives:

1) Prevent southern motor control panel from overturning

Upgrade Recommendations:

1) Anchor southern motor control to floor with four (4) 1/2-in. diameter kwik bolts

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	\$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	\$ 640

Accuracy of Estimate: ±30%

11.4 Beacon Hill Water Quality Administration Building/ Chlorination Facility and Sodium Hypochlorination Facility

11.4.1 Facility Description

Water Quality Administration Building

The Beacon Hill Water Quality Administration Building is located on the southeast corner of Spokane Street and 15th Avenue South. The original structure was built in 1956, and measured 34 ft. wide by 51 ft. long. The roof of the building was 2 x 6-in. T & G decking supported on wood beams. These roof beams were supported on the exterior walls and intermediate steel beams. The building is a two-story structure with a concrete foundation. The walls for the lower story are concrete basement walls and pumice block walls. The upper story floor is framed with 2 x 10-in. joists at 16-in. o.c. bearing on the concrete and block walls. The wood floor is anchored to the block walls with steel straps at 6-ft. o.c. The roof diaphragm is not positively connected to the masonry walls, causing stress to the wood members across the grain.

In 1973, an addition was completed to the original building. The addition was mainly built onto the north and west elevations of the building. The addition utilizes 2 x 6-in. stud wall construction with a masonry veneer, and wood framing for the roof and floor.

Sodium Hypochlorination Facility

The Beacon Hill Sodium Hypochlorination Facility is located about 300 ft. south of the Water Quality Administration Building. The building was designed in 1987, and constructed in 1988. It measures 35 ft. x 27 ft., and is of stud construction with brick veneer. The building has a 2-1/2 on 12 pitch plywood and asphalt roof supported on 2 x 10-in. joists spanning half-width at 2-ft. o.c. The structure has a 6-in. thick concrete slab foundation on grade.

11.4.2 Structure Evaluations

Water Quality Administration Building

The original construction documents showed 8-in. block walls with horizontal bond beams at about 48-in. o.c. The vertical reinforcing in the walls was not obtainable from the drawings. The block walls had a positive direct connection at the floor diaphragm, but not at the roof level. Several of the details appeared to stress the wood members across the grain. The block walls were doweled to the concrete foundation walls at 48-in. o.c. The exterior block walls were faced with a brick veneer that was positively attached to the block walls with steel ties.

The addition appears to be well constructed; however, from the drawings it appears that the shear capacity of the walls at the roof diaphragm level is lacking because of the presence of a continuous band of windows across the north and west elevations. There are drag struts over the windows, but there is no connection of the drag struts to the shear walls. The lateral force-resisting system of the addition and the existing portion of the original building should be investigated further.

According to the ATC-14 reinforced masonry building checklist, this structure has several weak links, mainly due to the lack of adequate vertical reinforcing and positive ties from the masonry walls to the roof diaphragm. In our opinion, a more in-depth engineering study of this building needs to be completed before the seismic risk can be accurately assessed. Based upon these preliminary observations, we rate the building structure as having a moderate vulnerability to seismic damage in a Class I event. For cost estimation purposes, we anticipate that seismic strengthening can be accomplished by bolting anchor straps with brackets 4-ft. o.c. from the roof to the masonry walls, and providing shear panels at the end of the drag struts.

<u>Sodium Hypochlorination Facility</u>

According to the construction plans, the roof rafters bear on a double 2 x 4-in. top plate nailed to the 2 x 4-in. stud wall. There is blocking between the joints. The facility has a built-up roof on 5/8-in. plywood sheathing. Roof joist-to-wall connections are not shown in the plans.

The ridge beam is supported at the center on a tube steel column which is fillet welded to a Simpson PC 44-16 post cap. There is also an interior longitudinal wall, full height with 2 x 4-in. studs at 16-in. o.c. under the ridge beam; however, details of the connection to the ridge beam are not specified in the plans. Nonetheless, the support provided by this beam is sufficient for a building of this size. The construction plans do not indicate any reinforcement of the masonry veneer or ties between the veneer and the stud wall. However, such ties are not required by the current edition of the Uniform Building Code.

11.4.3 Equipment and Contents Evaluations

__Water Quality Administration Building__

Lockers and Storage Bins

All of the tall lockers within the building should be bolted to the walls or the floor to prevent overturning. The parts storage racks should be bolted to the floor. The laboratory equipment appears to have a moderate vulnerability to damage, with a number of glassware and chemical items being stored on open shelves and table tops.

Chlorinator and Chlorine Analyzer

This equipment is located near the northeast corner of the lower portion of the building. The chlorinator cabinet is framed out of light weight plastic and is anchored in all four corners. Most of the piping connected to the equipment is plastic, which has some flexibility for movement. The steel piping, however, is not that flexible and could rupture during an earthquake. The reliability of this equipment could be improved by providing flexible couplings on the piping. We find this equipment to have a low vulnerability to seismic damage.

Portable Chlorine Analyzers

Two small analyzers are supported on a workmate. These units are subject to falling off of the workmate if not positively attached.

Chlorine Tanks

The chlorine tanks are on scale pits and are supported on two wooden saddles. It is possible that the tanks could slide off these saddles, causing the chlorine feeder lines to rupture. One method of solving this potential hazard is to strap the chlorine tanks around their center. The strap should have a ratchet and buckle system that is easy to de-couple. An alternative method is to construct removable reinforced bollards at each end of the tank rack, and tube steel rail headers paralleling the tank rack on both sides as shown in Figure 11.40-1.

The gas tanks in the laboratory are chained to the wall and have a low vulnerability to damage.

__Sodium Hypochlorination Facility__

All of the equipment within the Sodium Hypochlorination facility was inspected and observed to be properly restrained and anchored.

11.4.4 Summary and Conclusions

a) From this preliminary structural investigation, a few possible weak links have been identified in the Water Quality Administration Building structural design that need to be investigated further. The lateral system of the new addition at the roof level needs to be verified. After a more in-depth engineering analysis of some of the items listed in the structure section, some remedial strengthening of the building may be required.

For cost estimation purposes, it has been assumed that strengthening will be accomplished by using anchor straps with brackets to anchor the roof to the walls, and constructing shear panels at the ends of the drag struts.

The new Sodium Hypochlorination Facility is properly designed and detailed to remain functional following a Class I seismic event.

b) Most of the equipment within the Water Quality Administration Building has a low vulnerability to seismic damage. The one ton chlorine tanks should be secured to prevent rupture of the feeder lines and the chlorine analyzers should be strapped to their work bench. The lockers and storage bins should be anchored.

All equipment within the Sodium Hypochlorination Facility is properly restrained and anchored.

c) Table 11.4-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.4-2.

Table 11.4-1

**Summary of Preliminary Seismic Evaluations
Beacon Hill Water Quality Administration Building (High Priority)
and Sodium Hypochlorite Facility (High Priority)**

Item	Description	Class I	Class II	Comment
Water Quality Administration Building				
1	Structure	2	1	More analysis required
2	Chlorinators	1	1	Consider flexible couplings
3	Chlorine tanks & analyzers	2	1	Tie with strap
4	Building addition	2	1	Investigate lateral capacity
5	Lockers and storage bins	2	1	Anchor
Sodium Hypochlorination Facility				
1	Structure	1	1	O.K.
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.4-2

Cost Estimate

Location: Beacon Hill Water Quality Administration Building and Chlorination Facility

Date Constructed: 1956 (Original) and 1973 (Addition)

Priority: High

Seismic Strengthening Objectives:

1) Strengthen ties from roof to masonry walls
2) Improve shear capacity of walls in building addition containing continuous band of windows
3) Secure lockers and storage racks
4) Secure chlorine tanks and analyzers

Upgrade Recommendations:

1) Provide anchor straps and brackets to connect roof to walls
2) Construct shear panels at the ends of the drag struts
3) Bolt lockers and storage racks to walls or floor
4) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides. Secure chlorine analyzers with a strap.

Additional Investigation Required:

1) Detailed review of lateral force resisting system of structure
2) Evaluate lack of vertical reinforcing on seismic performance of structure
3) Verify block wall-to-roof diaphragm connection

Assumption: Construction Performed Outside (Except for securing lockers, racks and chlorine analyzers)

Cost Estimate:

1)	Additional Investigation	\$ 5,000
2)	Engineering	\$ 25,600
3)	Construction Engineering	19,700
4)	Construction	49,200
	Subtotal:	99,500
5)	Sales Tax (8.1%)	4,000
	Total:	\$103,500

Accuracy of Estimate: ±40%

11.5 Bitter Lake Pump Station and Chlorination Building

11.5.1 Facility Description

The Bitter Lake facility, constructed in 1958, is near Linden Ave. N. and N. 143rd St. and supplies water to the 580 zone. The structure consists of a cast-in-place reinforced concrete partial basement area and a one-story building above on continuous footings. The building measures approximately 30 ft. by 60 ft.. The building is "L" shaped in plan. The roof diaphragm consists of 2 x 6-in. T & G decking over three (3) 2 x 12-in. built-up wood beams. The roof perimeter is then supported on a short 2 ft. 6-in. high, 2 x 6-in. stud wall above an 8-in. c.m.u. wall (c.m.u. walls are faced with 4-in. thick brick veneer) with a height of about 7 ft. The roof beams have a 2:12 pitch, are spaced at 7 ft. 6-in. o.c., and are supported by a ridge beam at the center of the building.

11.5.2 Structure Evaluations

From the original construction documents, the 8-in. c.m.u. exterior walls are reinforced horizontally with K-web reinforcing at 24-in. o.c. One continuous bond beam with two (2) #5 bars is typically located at the wall mid-height. Little vertical reinforcing is provided in the c.m.u. walls according to the drawings. Since the c.m.u. walls do not extend to the roof diaphragm or have a continuous horizontal bond beam at the top, the walls lack sufficient out-of-plane support and are subject to failure during an earthquake. The short 2 ft. 6-in. walls above the masonry walls appear to lack sufficient shear strength due to the small shear capacity of horizontal wood sheathing applied to the stud walls.

The center 11 ft. high c.m.u. wall that partitions the chlorination facilities and the pump station lacks positive anchorage to the roof diaphragm as well as adequate reinforcing. We feel that in a Class I type earthquake this structure will not provide acceptable seismic performance, thus, we judge this structure as having a high vulnerability to seismic damage.

11.5.3 Equipment and Contents Evaluations

Pumps

Three pumps are contained within the building. These pumps are well anchored and have a low vulnerability to damage.

Electrical Control Panels

The anchorage of the motor control panel on the concrete platform should be verified.

Chlorinator and Chlorine Analyzer

This equipment is located in the north end of the pump station. The chlorinator cabinet is framed out of light weight plastic and is anchored in all four corners. Although most of the piping and conduits are not flexible, the chlorine piping has some flexibility that allows some movement. The reliability of this equipment could be increased by providing flexible piping, at least to the upper portion of the units. We find this equipment to have a low vulnerability to damage.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method to prevent this potential hazard is to secure the tanks with a strap around their center. Alternatively, removable reinforced bollards can be installed at the ends of the tank rack, along with tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

11.5.4 Summary and Conclusions

a) From this preliminary structural investigation, several weak links have been identified regarding the seismic resistance of this building. The improvements that could be made to increase the seismic resistance of this structure are:

   1) Remove the existing 2 ft. 6-in. walls that connect to the roof diaphragm and extend the c.m.u. block walls to the roof, adding new vertical reinforcing to the existing walls and new horizontal bond beams to the new block walls.

   2) Provide new connections between the c.m.u. wall and the roof structure and install a new plywood diaphragm over the existing 2 x 6-in. T & G decking.

   3) Add vertical reinforcing to the interior portion of the walls.

b) Most of the chlorination equipment has a low vulnerability to seismic damage. The chlorine tanks should be strapped together as specified above.

c) The anchorage of the motor control equipment should be verified, and provided if lacking.

d) Table 11.5-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.5-2.

Table 11.5-1

Summary of Preliminary Seismic Evaluations
Bitter Lake Pump Station and Chlorination Building (High Priority)*

Item	Description	Class I	Class II	Comment
1	Structure	3	2	No continuity in in c.m.u. walls and lack of reinforcement
2	Pumps (Medium Priority)	1	1	Well anchored
3	Electric control panel	2	1	Verify anchorage
4	Chlorine equipment (High Priority)	2	1	Provide hold-down strap
5	Chlorinators	1	1	Consider flexible couplings

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

* High priority rating of facility is due to treatment rather than pump station function.

Table 11.5-2

Cost Estimate

Location: Bitter Lake Pump Station

Date Constructed: 1958

Priority: High

Seismic Strengthening Objectives:

1) Anchor roof diaphragm to walls
2) Vertically reinforce existing c.m.u. walls
3) Provide horizontal stabilization to tops of existing c.m.u. walls
4) Shear strengthen the roof diaphragm
5) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing
6) Verify anchorage of motor control panel

Upgrade Recommendations:

1) Remove existing 2 ft. 6-in. walls and extend c.m.u. walls to roof, including installation vertical reinforcement and bond beams
2) Add vertical reinforcement to existing c.m.u. walls
3) Install plywood diaphragm on T & G decking
4) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides
5) Verify/add anchorage to motor control panel, as necessary

Assumption: Construction Performed Outside (Except for securing control panel and chlorine tanks)

Cost Estimate:

1)	Engineering	$ 28,700
2)	Construction Engineering	16,000
3)	Construction	53,000
	Subtotal:	97,700
4)	Sales Tax (8.1%)	4,300
	Total:	$102,000

Accuracy of Estimate: ±35%

11.6 Bothell Way Pump Station

11.6.1 Facility Evaluations

The Bothell facility was constructed in 1956 and is located on the N.E. corner of Bothell Way and E. 82nd St. The facility serves as a booster pump station for the 420 zone. The structure is a cast-in-place, reinforced concrete below grade building on continuous footings. The building measures 23 ft. by 34 ft. The concrete slab roof is supported by three transverse concrete beams and is nearly 12 ft. above the floor slab. The roof slab has a removable portion that measures 19 ft. by 10 ft. with concrete roof beams framed around the opening. No evidence of settling or wall cracking was observed in the structure.

11.6.2 Structure Evaluations

From the original construction documents, the concrete perimeter walls are generally 10-in. thick and are provided with #6 bars at 6-in. o.c. vertical and #4 bars at 10-in. o.c. horizontal. The horizontal reinforcing is slightly less than that required by the current ACI code. The connection of the walls to the slab diaphragm appear to be adequate to resist seismic forces. The walls are not doweled to the continuous footings, however, a 2 x 4-in. shear key is provided which will probably be sufficient. The large roof access opening in the diaphragm is of some concern, however, this opening is bounded by concrete beams which should transfer the lateral load through the opening and eventually into the shear walls. In conclusion, we feel that in a Class I earthquake, this structure will experience minor visible cracking or yielding of a few members which will not affect the structural integrity of the building.

11.6.3 Equipment and Contents Evaluations

Pumps

One pump is contained within the pump station. The pump rests on a large concrete pedestal and is anchored with 1 1/2-in. diameter anchor bolts. This equipment is considered to have a low vulnerability to earthquake damage.

Electrical Control Panels

The motor control panel that operates the pump appears to have a low vulnerability to seismic damage due to its relatively large width.

Air Conditioning Unit

The air conditioning unit located at the N.E. corner of the building appears not to be properly anchored to the concrete pedestal.

11.6.4 Summary and Conclusions

a) From this preliminary investigation, it appears that this structure will experience minor structural damage. Remedial strengthening is not required.

b) The anchorage of the motor control panel to the floor slab should be verified. The air conditioning unit should be anchored to the concrete pedestal or to the concrete wall.

c) Table 11.6-2 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.6-2.

Table 11.6-1

Summary of Preliminary Seismic Evaluations
Bothell Way Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panel	3	2	Verify/supply anchorage
4	A/C unit	2	1	Supply anchorage

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.6-2

Cost Estimate

Location: Bothell Pump Station

Date Constructed: 1956

Priority: High

Seismic Strengthening Objectives:

1) Prevent air conditioning unit from overturning

Upgrade Recommendations:

1) Bolt air conditioner to concrete pedestal or concrete wall

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 200
2)	Construction Engineering	0
3)	Construction	700
	Subtotal:	900
4)	Sales Tax (8.1%)	60
	Total:	$ 960

Accuracy of Estimate: ±30%

11.7 Boulevard Well Pump Station and Treatment Facility

11.7.1 Facility Description

This facility, located at 20th Avenue S. and S. 128th Street, supplies water to West Side Pipeline No. 4 and also serves as a water treatment facility. Constructed in 1987, the building is a single-story brick and mortar structure measuring 35 ft. by 42 ft. The roof consists of prefabricated trusses at 24-in. o.c. with a plywood roof diaphragm. Several interior non-bearing walls serve to separate the treatment rooms from the pump rooms.

11.7.2 Structure Evaluations

According to the construction drawings, the exterior bearing walls consist of 6 x 12-in. structural brick reinforced with #5 vertical bars at 48-in. o.c., and two #5 horizontal bars at 48-in. o.c. The walls also have horizontal joint reinforcing at every fourth course. All of the vertical wall reinforcing is doweled into the concrete perimeter foundation. The trusses bear on a double 2 x 8-in. top plate that is anchor-bolted to the top brick bond beam with bolts at 24-in. o.c. The trusses are attached to the wood plate with a Simpson H1, which appears to be a seismically adequate connection. The exterior walls of the building have only a few penetrations, which suggests that the shear stress in the masonry is not critical with the reinforcing provided. The interior walls within the structure are well anchored and provide additional stability. The interior walls are attached positively for support to the bottom chord of the trusses with an adequate seismic detail.

With these considerations, we find that this structure has a low vulnerability to a Class I earthquake.

11.7.3 Equipment and Contents Evaluations

Chemical Tanks

The chemical tanks in the chemical room are set into a concrete saddle and are anchored with straps to the saddle. We find this equipment to have a low vulnerability to damage.

Pumps and Electrical Control Panels

The pumps and piping are very well anchored to large concrete footings and are considered to have low vulnerability to damage. The chlorine, well monitoring and pump control equipment are all well anchored except for the Westinghouse transformer XFMR T-1. This unit should be anchored. Anchorage of the motor control center consoles could be verified.

11.7.4 Summary and Conclusions

a) This structure has recently been designed and constructed and we feel only minor structural damage will occur in this building from a Class I earthquake.

b) The equipment in the structure has a low vulnerability to seismic damage with exception of the electrical transformer T-1 which needs to be anchored.

c) Table 11.7-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.7-2.

Table 11.7-1

Summary of Preliminary Seismic Evaluations
Boulevard Well Pump Station and Treatment Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Chemical tanks	1	1	Well supported
3	Pumps	1	1	Well anchored
4	Motor control units	2	1	Verify anchorage
5	Transformer T-1	2	1	Anchorage required

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.7-2

Cost Estimate

Location: Boulevard Well Pump Station and Treatment Facility

Date Constructed: 1987

Priority: High

Seismic Strengthening Objectives:

1) Prevent Westinghouse transformer unit XFMR T-1 from overturning

Upgrade Recommendations:

1) Bolt XFMR T-1 unit to floor

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: ±30%

11.8 Broadway Pump Station and Chlorination Building

11.8.1 Facility Description

This facility, located near Nagle Place and Olive Street, is a combined pump station and chlorine facility constructed in 1957. The chlorine facility serves the Lincoln Park reservoir. The pump station supplies water to the 520 zone. The building has a lower basement area that measures about 26 ft. by 30 ft. with 12-in. thick doubly-reinforced concrete walls and a slab on grade. This area is used for housing pump and piping equipment. An intermediate concrete floor slab area supported on concrete beams above the area just described is used to support the electrical motor control equipment. The building measures 30 ft. by 40 ft. and has many window and door openings.

11.8.2 Structure Evaluations

The chlorine rooms at the north end of the building are separated from the pump room by an 8-in. block wall that extends full height to the 2 x 6-in. T & G roof diaphragm. This wall only has K-web reinforcing at every third course and no connection at the roof diaphragm is detailed in the construction drawings. The building has a single story block wall and 2 x 6-in. framed wood wall with 4 x 12-in. roof beams supporting the 2 x 6-in. T & G sheathing. Sandstone facing, 4-in. thick, is securely attached to the 8-in. block walls. While inspecting the chlorine facility rooms, a few wide cracks were noticed in the block walls which suggests past wall distress, possibly due to seismic action.

According to the construction drawings, the exterior block bearing walls have K-web horizontal reinforcing every third course and no vertical reinforcing. The block walls are dowled to the concrete foundation walls with #5 bars at 4-ft. o.c. Two types of wall construction exist in this building: 1) concrete block walls extending from the foundation to the roof diaphragm; and 2) concrete block walls extending to about 4 ft. above the foundation with a 2 x 6-in. stud wall continuing to the roof diaphragm. The block walls that extend to the roof appear to only be tied-in where a 4 x 12 in. roof beam bears on the wall. These full height block walls should be anchored to the roof diaphragm with steel straps about every 4 ft.

The stud walls appear to have wood sheathing attached to the exterior of the walls. This sheathing supplies some shear wall resistance but how much is unclear. The shear resistance of these walls is important since long lengths of the building are framed in this manner. The interior of the building is finished with acoustical tile on 1 x 3-in. wood sheathing. If deflection of the walls due to seismic action occurs, this tile could be damaged and could separate from the walls and ceilings. Given these considerations, we rate this

structure as having a moderate vulnerability to a Class I earthquake and recommend that additional analyses be completed to predict more accurately the building's performance.

11.8.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This equipment is located in the northeast half of the building. The chlorinator cabinet is framed out of light weight plastic and is anchored in all four corners. Although most of the piping and conduits are not flexible, the chlorine piping has some flexibility that allows some movement. The reliability of this equipment could be increased by providing flexible piping, at least to the upper portion of the units. Overall, we find this equipment to have a low vulnerability to damage.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method to prevent this potential hazard is to secure the tanks with a strap around their center. Alternatively, removable reinforced bollards can be installed at the ends of the tank rack, along with tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

Electrical Equipment

The electrical motor control panels within the pump station appear to be anchored to the intermediate concrete floor slab; however, this should be verified. The telemetry equipment, attached to the piping and pump equipment, is well secured to this equipment and is considered to have a low vulnerability to damage in an earthquake.

Pumps and Piping Equipment

The pumps and piping equipment within the pump station are well anchored to the concrete slabs and beams and are considered to have a low vulnerability.

11.8.4 Summary and Conclusions

a) The structure has some weak links, including the shear capacity of the stud walls, the reinforcement of the masonry walls, and the roof-to-wall connection. It is recommended that additional investigations be performed to determine the existing seismic resistance and load paths for this building. The addition of a plywood diaphragm

would increase seismic resistance, as well as positively connecting the block walls to the roof diaphragm.

b) Most of the chlorination equipment has a low vulnerability to seismic damage. The chlorine tanks should be strapped together as specified above.

c) The electrical and telemetry equipment was generally found to be well anchored. Anchorage of the electrical motor control panels should be verified. The chlorine tanks should be secured to prevent rupture of the feeder lines.

d) Table 11.8-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.8-2.

Table 11.8-1

Summary of Preliminary Seismic Evaluations
Broadway Pump Station and Chlorine Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Inadequate connection between walls and roof
2	Chlorine tanks	2	1	Provide hold-down strap
3	Chlorinator and Analyzer	1	1	Consider flexible couplings
4	Pumps & Piping	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.8-2

Cost Estimate

Location: Broadway Pump Station and Chlorine Facility

Date Constructed: 1957

Priority: High

Seismic Strengthening Objectives:

1) Improve wall-to-roof connections
2) Strengthen shear resistance of roof diaphragm
3) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing

Upgrade Recommendations:

1) Connect full height block walls to roof with straps 4-ft. o.c.
2) Install plywood roof diaphragm
3) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides

Additional Investigation Required:

1) Conduct detailed analysis to more accurately predict seismic performance of structure

Assumption: Construction Performed Outside (Except for securing chlorine tanks)

Cost Estimate:

1)	Additional Investigation	$ 5,000
2)	Engineering	21,300
3)	Construction Engineering	9,500
4)	Construction	41,700
	Subtotal:	77,500
5)	Sales Tax (8.1%)	3,400
	Total:	$80,900

Accuracy of Estimate: +35%

11.9 Burien Pump Station

11.9.1 Facility Description

The Burien facility, located on the corner of S. 146th Street and 8th Ave S., supplies water to the 575 zone and the Myrtle and Beverly Park water tanks. The structure consists of a cast-in-place reinforced concrete partial basement area and a one-story building above supported on continuous footings. The building measures approximately 28 ft. by 36 ft. and was constructed in 1954. The roof diaphragm consists of 2 x 6-in. trusses at 16-in. o.c., with 1 x 6-in. sheathing at 8-in. o.c. It has a cedar shake roof, whose perimeter is supported on masonry walls. The interior ceiling of the building is acoustical material attached to furring strips.

11.9.2 Structure Evaluations

From the original construction documents, the exterior structural walls are constructed out of half-high, 8 x 16-in. mediterranean block which is reinforced horizontally at 16-in. o.c. with "K-web" reinforcing. Vertical reinforcing exists only at door and window openings and the doweling to the foundation is uncertain. The top bond beam is reinforced with one #3 bar. The roof trusses bear on 2 x 6-in. plates bolted to the top course of masonry; however, positive direct anchorage does not appear to be provided by the "trip-l-grip" anchor provided in the connection details. It also appears that these type of anchors do not prevent cross grain bending in the wood plate as required by the current Uniform Building Code. The roof diaphragm as it exists is weak and will allow yielding and cracking of the masonry walls. This may be evidenced by the many stress cracks that are observed in the structure which may have been caused by a past earthquake. The acoustical ceiling would be damaged in a Class I event because of the weak roof diaphragm.

The diaphragm action of this building could be improved by removing the existing shake roof and placing a 3/8-in. plywood diaphragm over the existing roof members.

The building has several door or window openings in each elevation; however, the masonry wall shear stress caused by a Class I earthquake does not appear to be significant. The unreinforced masonry walls lack a positive tie-in to the roof diaphragm as well as proper reinforcing. Not much can be done about the inadequate reinforcing without considerable cost; however, significant damage can be minimized by tying the building together.

In a Class I earthquake, this structure as it exists will not provide acceptable seismic performance. We rate this structure as having a moderate vulnerability to seismic damage.

11.9.3 Equipment and Contents Evaluations

Pumps

The three pumps contained within the building are well anchored and have a low vulnerability to damage.

Electric Control Panels

The anchorage of the motor control panel on the concrete platform should be verified and provided if absent.

11.9.4 Summary and Conclusions

In this preliminary structural investigation, several weak links were identified in the seismic resistance of this building. The seismic resistance of this structure could be improved by the following actions:

1) Remove the existing roofing and install a new plywood diaphragm to the existing wood roof members. Anchor 2 x 6-in. roof trusses to the masonry walls with steel straps at 4 ft. o.c. A method for reinforcing the existing the 8-in. block wall should be investigated.

2) The anchorage of the motor control equipment should be verified and provided if absent.

3) Table 11.9-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.9.2

Table 11.9-1

Summary of Preliminary Seismic Evaluations
Burien Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Unreinforced masonry walls; strengthen roof and walls
2	Pumps	1	1	Well anchored
3	Electric control Panels	2	1	Verify anchorage

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.9-2

Cost Estimate

Location: Burien Pump Station

Date Constructed: 1954

Priority: Medium

Seismic Strengthening Objectives:

1) Strengthen roof diaphragm
2) Provide positive anchorage of roof to masonry walls

Upgrade Recommendations:

1) Add 3/8-in. plywood roof diaphragm
2) Tie roof trusses to masonry walls with steel straps 4-ft. o.c.

Assumption: Construction Performed Outside

Cost Estimate:

1)	Engineering	\$20,500
2)	Construction Engineering	8,700
3)	Construction	30,000
	Subtotal:	59,200
4)	Sales Tax (8.1%)	2,400
	Total:	\$61,600

Accuracy of Estimate: ±35%

11.10 Dayton Ave. Pump Station

11.10.1 Facility Description

The Dayton facility, located on the N.E. corner of Dayton Ave. N. and N. 145th St., supplies water to the high 650 zone. The pump chamber is a precast concrete vault placed upon a concrete slab on grade. The vault precast walls appear to be joined at mid-height above the slab, with a total wall height of 8 1/2 ft. The roof also appears to be a precast concrete slab about 12-in. thick. The only opening in this vault is the north door entrance.

11.10.2 Structure Evaluations

The connection of the precast concrete elements was not apparent, but since this structure was built in 1977 it is assumed that all of the attachments are adequate. It appears that the perimeter walls may be doweled into a thickened slab that was placed within the chamber. Thus, resistance to lateral loads transferred to the perimeter shear walls by the rigid concrete roof diaphragm seems to be sufficient for a small 10 ft. by 20 ft. structure such as this. Based upon this preliminary investigation, we feel that in a Class I earthquake this structure would suffer only minor structural damage.

11.10.3 Equipment and Contents Evaluations

Pumps

Two pumps are contained within the pump chamber. Both pumps rest on concrete pedestals and are anchored with anchor bolts. This equipment is considered to have a low vulnerability to earthquake damage.

Electrical Control Panels

There are four electric panels that are attached to the precast concrete walls with four bolts at each corner. The electrical equipment is enclosed in rigid sheet metal cabinets. These items are rated as having a low vulnerability to damage from earthquakes.

11.10.4 Summary and Conclusions

a) Since this structure was built in 1977, it is reasonable to assume that the precast concrete element connections are adequate. Therefore, this structure is determined to be properly designed to withstand the Class I earthquake with only minor structural damage.

b) The equipment is rigidly attached to the walls of the pump chamber. Since the walls are also rigid, there will be little displacement and thus only negligible damage is expected.

c) Table 11.10-1 summarizes the results of the seismic evaluations.

Table 11.10-1

Summary of Preliminary Seismic Evaluations
Dayton Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panel	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.11 Eastgate Pump Station

11.11.1 Facility Description

The Eastgate facility is located within Eastgate park just off of Newport Way. With construction being completed in 1989, the facility will be used as a booster pump station to fill Eastgate Reservoir as well as provide backup service for peak hour operation. The structure is a cast-in-place reinforced concrete below-grade building on a 2-ft. thick mat foundation. The building measures 30 ft. by 47 ft. The 8-in. concrete roof slab is supported by three longitudinal concrete beams and is nearly 13 ft. above the lower floor slab. The roof slab has four removable portions that measure 4 ft. by 4 ft., and one penetration that measures 7 ft. by 4 ft. Concrete roof beams are framed near all openings. No evidence of settling or cracking was observed.

11.11.2 Structure Evaluations

From the original construction documents, the concrete perimeter walls range from 10 in. to 14 in. and minimum reinforcement areas are in compliance with the current ACI code. The connection of the walls to the slab diaphragm are adequate to resist seismic forces. The walls are adequately doweled to the mat foundation in each face. The access openings in the roof diaphragm appear to be adequately reinforced to transfer the lateral load through the openings and eventually into the shear walls due to the perimeter concrete beam which frames the openings. In general, all of the concrete beams and columns are well reinforced and are provided with stirrups. We conclude that this structure will experience only minor visible cracking in a few members from a Class I earthquake. The structural integrity of the building should be sustained. The building has a low vulnerability to seismic damage.

11.11.3 Equipment and Contents Evaluations

Pumps

The three pumps contained within the pump station rest on the large concrete mat foundation that is 2-ft. thick and is anchored to the concrete with 1-in. diameter anchor bolts at 5-in. o.c. around the circular base. The pumps stand about 8 ft. tall and have flexible power and telemetry lines. This equipment has a low vulnerability to earthquake damage.

Electric Control Panels

The motor control panels are located at the west side of the room on the upper slab area. All motor control units appear to have a low vulnerability to seismic damage.

Air Control Equipment

The dehumidifier and heating units are anchored to the concrete wall and all the electrical conduits have flexible lines. This equipment is considered to have a low vulnerability to seismic damage.

Piping

The 24-in. diameter piping in the pump station is well connected to the floor slab and supported by concrete saddles with a neoprene pad between the pipe and saddle. At the penetrations through which the piping exits the pump station, the space between the piping and the concrete wall is provided with a seismic-resistant connection. Virtually all power and instrumentation connections in the facility are flexible, and the building contains the current state of the art in earthquake resistant design features.

11.11.4 Summary and Conclusions

a) From this preliminary investigation of the original construction documents, it appears that this structure will experience only minor structural damage in a Class I earthquake. Nonstructural damage will also be minor.

b) The equipment anchorage and supporting lines have good earthquake resistant design and, thus, low vulnerability to damage.

c) Table 11.11-1 summarizes the results of the seismic evaluations.

Table 11.11-1

Summary of Preliminary Seismic Evaluations
Eastgate Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panels	1	1	Well anchored
4	A/C equipment	1	1	Well anchored
5	Piping	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.12 Fairwood Pump Station

11.12.1 Facility Description

This facility is located just northwest of the Lake Youngs Control Works building. Constructed in 1968, it supplies water to the higher 630 zone. The building is a single-story block and mortar structure measuring 22 ft. by 26 ft. with an 8-in. perimeter foundation wall and a 16-in. footing. The structure is composed of 8-in. cinder block walls extending above the concrete perimeter walls to the roof diaphragm. Two interior 16-in. square block pilasters support a single 4 x 16-in roof beam in the center of the building. The roof diaphragm is approximately 15 ft. above the slab on grade.

The construction drawings show that roof loads are transferred through 3 x 6-in. T & G decking to a center 4 x 16-in. beam. The exact connection of the roof diaphragm to a supporting steel tee was not obtainable from the drawings or the field inspection. However, newly-fabricated roof trusses have recently been installed over the existing parapet walls. The trusses were toe nailed to a wood plate on top of the parapet walls. The possibility of cross-grain bending may exist in the connection to the roof diaphragm since that particular detail is not clear. Further investigation is required to determine the adequacy of this connection.

11.12.2 Structure Evaluations

According to the construction drawings, the 8-in. hollow block exterior walls have horizontal K-web joint reinforcing at each third course. There is no detailing of vertical reinforcement. The only opening in the structure is the 6 ft. door opening in the northwest side of the building. No trim reinforcing around the door opening was called for in the drawings. Also, no top bond beam was called out in the drawings near the roof diaphragm that contained reinforcing. The 16-in. square block pilasters are reinforced with four #5 bars vertical and #2 ties at 12-in. o.c.

An evaluation made using the ATC-14 rapid evaluation check list revealed several potential weaknesses due mainly to the lack of reinforcing and positive anchorage of the walls to the roof diaphragm. With these considerations, we rate this structure as having a moderate vulnerability to a Class I earthquake. The existing block walls are expected to experience extensive cracking in such an event.

11.12.3 Equipment and Contents Evaluations

Electrical Control Equipment

The electrical motor control panels on the concrete floor slab appear to be anchored to the slab; however, this should be verified. If they are anchored, their vulnerability to damage will be low. All of the other electrical panels attached to the walls at the upper slab level are well anchored and have low vulnerability to seismic damage. The telemetry equipment, attached to the pipes within the pump station, also has low vulnerability to seismic damage.

Pump Equipment

Two pumps exist within the pump station. They are well anchored and have a low vulnerability to seismic damage.

11.12.4 Summary and Conclusions

a) A more detailed analysis and inspection should be completed to determine the seismic integrity of the T & G diaphragm and to verify that a positive, seismic connection to the steel tee exists. Since no top bond beam with reinforcing exists according to the drawings, steel angles should be installed at the T & G diaphragm level with thru bolts in the block walls. This would serve to tie the building together.

b) The electrical control equipment appears to be anchored adequately. Verification of anchorage for the motor control units should be completed.

c) Table 11.12-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.12-2.

Table 11.12-1

Summary of Preliminary Seismic Evaluations
Fairwood Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Potentially weak wall-to-wall and roof connections
2	Pumps	1	1	Well anchored
3	Electric control panel	1	1	Verify anchorage

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.12-2

Cost Estimate

Location: Fairwood Pump Station

Date Constructed: 1968

Priority: Medium

Seismic Strengthening Objectives:

1) Determine the strength of the roof diaphragm connection to the supporting steel tee
2) Tie roof diaphragm to block walls
3) Verify electric control panel anchorage

Upgrade Recommendations:

1) Tie-in the top course of block walls to roof diaphragm
2) Verify electric control panel anchorage and upgrade, as required

Additional Investigation Required:

1) Conduct investigation of connection between roof diaphragm and supporting steel tee

Assumption: Construction Performed Outside (Except for anchorage of control panel)

Cost Estimate:

1)	Additional Investigation	$3,000
2)	Engineering	10,900
3)	Construction Engineering	6,100
4)	Construction	20,000
	Subtotal:	40,000
5)	Sales Tax (8.1%)	1,600
	Total:	$41,600

Accuracy of Estimate: ±35%

11.13 First Hill Pump Station

11.13.1 Facility Description

The First Hill facility located on the N.W. corner of East Jefferson St. and 12th Ave., is used to supply water to the First Hill 520 zone, which doesn't have storage. The structure, built in 1984, is a cast-in-place reinforced concrete below grade building on a mat foundation. The building measures 20 ft. wide by approximately 45 ft. long. The concrete slab roof is supported by four transverse concrete beams and is nearly 11 ft. above the floor slab. The roof slab has three removable portions that measure 4 ft. by 5 ft. with concrete roof beams framed near the openings. No evidence of settling or wall cracking was observed in this building.

11.13.2 Structure Evaluations

From the original construction documents the concrete perimeter walls are 10-in. thick, and are provided with #4 bars at 11 or 15-in. o.c. vertical each face and #4 bars at 18-in. horizontal each face, which comply with the ACI code current at the time of design. The connection of the walls to the slab diaphragm are adequate to resist seismic forces. The walls are securely doweled to the mat foundation. The access openings in the roof diaphragm are adequately reinforced to transfer the lateral load through the opening and eventually into the shear walls. In conclusion, we feel that in a Class I-type earthquake this structure will experience minor visible cracking in a few members. The structural integrity of the building shouldn't be affected.

11.13.3 Equipment and Contents Evaluations

Pumps

The pumps contained within the pump station rest on large concrete pedestals and are anchored adequately. This equipment is considered to have a low vulnerability to earthquake damage.

Electrical Control Panels

The motor control panels are located in a separate room at the north end of the facility. They all appear to have a low vulnerability to seismic damage as may be expected from their recent construction; however, anchorage of the motor control panels should be verified and the panels bolted to the floor if they are not adequately anchored.

Air Conditioning Unit

The air conditioning unit located at the north end of the building is suspended from the concrete roof by 1/2-in. diameter rods and has a low vulnerability to seismic damage.

11.13.4 Summary and Conclusions

a) From this preliminary investigation, it appears that this structure will experience only minor structural damage from a Class I earthquake. Nonstructural damage will also be minor.

b) The anchorage of the motor control panels to the floor slab should be verified and provided if lacking.

c) Table 11.13-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.13-2.

Table 11.13-1

Summary of Preliminary Seismic Evaluations
First Hill Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panel	1	1	Verify anchorage
4	A/C unit	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.13-2

Cost Estimate

Location: First Hill Pump Station

Date Constructed: 1984

Priority: High

Seismic Strengthening Objectives:

1) Prevent motor control panels from overturning

Upgrade Recommendations:

1) Verify/anchor control panels

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: +30%

11.14 Foy Pump Station

11.14.1 Facility Description

The Foy facility, located at the northeast corner of 5th Avenue N.E. and N.E. 145th Street, supplies water to the 580 zone and the water tank located near Fremont Avenue N. and N. 195th Street. The structure is cast-in-place reinforced concrete on continuous footings, constructed in 1933. The building measures 34 ft. by 42 ft.. The concrete slab roof is supported by three transverse concrete beams and is nearly 15 ft. above the interior slab on grade. The original window openings have been filled in with c.m.u. block.

11.14.2 Structure Evaluations

From the original construction documents, the 8-in. exterior concrete perimeter walls are generally reinforced with #3 bars at 12-in. o.c., which is slightly less than that required by the current ACI code. From a preliminary analysis of this structure, visible cracking or yielding of a few members may be expected to in a Class I earthquake; however, the structural integrity of the building will not be seriously impacted. The fact that the openings have been filled in with block increases the lateral stability of the entire structure. In our opinion, this structure has a moderate vulnerability to seismic damage in a Class I earthquake.

11.14.3 Equipment and Contents Evaluations

Pumps

The three pumps contained within the pump station are well anchored on concrete pedestals and are considered to have a low vulnerability to earthquake damage.

Electrical Control Panels

There are several electric panels that are located at the north end of the building. The electrical equipment is enclosed in rigid sheet metal cabinets. Due to the small width of the eastern-most control panel, we recommend that the panel be braced to the north wall to prevent overturning during an earthquake.

11.14.4 Summary and Conclusions

a) From this preliminary investigation, because #3 bars provide the main reinforcing in the concrete perimeter walls, a Class I earthquake may produce flexure in the walls such that the structure will experience some cracking and spalling of the masonry infill; however, the structural integrity should not be seriously impacted.

b) Brace the eastern-most control panel to the north wall

c) Table 11.14-1 summarizes the results of the seismic evaluations. An estimate of remedial upgrade costs is presented in Table 11.14-2.

Table 11.14-1

Summary of Preliminary Seismic Evaluations
Foy Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Some wall cracking will occur
2	Pumps	1	1	Well anchored
3	Electric control panel	3	2	Provide brace

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.14-2

Cost Estimate

Location: Foy Pump Station

Date Constructed: 1933

Priority: Medium

Seismic Strengthening Objectives:

1) Prevent eastern-most electrical control panel from overturning

Upgrade Recommendations:

1) Bolt eastern-most electrical control panel to north wall

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: +30%

11.15 Green Lake Pump Station

11.15.1 Facility Description

The Green Lake Pump Station facility, located on 12th Avenue N.E. and N.E. 73rd Street, supplies water to the 520 zone. The below-grade structure, built in 1933, consists of cast-in-place reinforced concrete basement walls on continuous footings. The building measures 10 ft. by 14 ft. The concrete slab roof is supported by two transverse steel beams and is 8 ft. above the interior slab on grade. The roof diaphragm has a penetration for pump maintenance that is built within the steel beams supporting the roof slab.

11.15.2 Structure Evaluations

The original construction documents did not provide enough detail to determine the reinforcing in the perimeter walls; however, during the inspection many hair line cracks were noted in the concrete walls and ceiling. This suggests that the walls probably have minimal reinforcing, if any. It appears that some settlement has occurred in the southwest corner of the building. From this very preliminary investigation, it is felt that a Class I-type earthquake will impart some visible cracking in the walls and ceiling; however, damage to the structure overall should be minor to moderate. We rate this structure as having a low vulnerability to seismic damage.

11.15.3 Equipment and Contents Evaluations

Pumps

Two pumps are contained within the building. These pumps are well anchored and have a low vulnerability to damage.

Electrical Control Panels

The few electrical control panels appear to be well anchored to the concrete walls and have low vulnerability.

11.15.4 Summary and Conclusions

a) From this preliminary structural investigation we find the structure to have low vulnerability to seismic damage.

b) The electrical equipment is well anchored and has a low vulnerability to seismic damage.

c) Table 11.15-1 summarizes the results of the seismic evaluations.

Table 11.15-1

Summary of Preliminary Seismic Evaluations
Green Lake Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	Some wall cracking will occur
2	Pumps	1	1	Well anchored
3	Electric control panel	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.16 Green Lake Chlorination Building and Gate House

11.16.1 Facility Description

The Green Lake facility is located at 12th Ave. N.E. and N.E. 73rd St., west of the Green Lake Reservoir. The original Gate House was built in 1911 and the current chlorine facility was added in 1957. The chlorination facility treats the water from the Green Lake Reservoir. The Gate House measures 24 ft. wide by 38 ft. long. The walls consist of 13 in. brick masonry extending to about 14 ft. 5 in. above the interior intermediate concrete floor slab. The roof is framed with pitched steel trusses at about 8 ft. on center with intermediate steel "I" sections placed above the trusses. A concrete roof slab is placed over the trusses that varies in thickness from 3 1/2 in. to 6 in. and is reinforced with a wire mesh.

The structure has a basement which is approximately 13 ft. from the slab on grade to the top of the intermediate floor slab. The intermediate floor slab consists of a 4 1/2-in. thick reinforced concrete slab supported on steel "I" beams and steel columns. The original intermediate floor slab area covered all but a 12 ft. by 26 ft. area. When the chlorine rooms were added to the original slab opening, this opening was reduced to 12 ft. by 7 ft. The foundation at the ground level is 18-in. thick and widens to 24-in. thick at the base with a 3-ft. wide continuous perimeter footing. All of the window openings have been filled in with block.

11.16.2 Structure Evaluations

According to the original construction drawings, the walls of the Gate House are 13-in. thick, unreinforced brick extending to about 12 ft. above the perimeter foundation. On top of this brick wall, the decorative clay tile extends 3 ft. to meet the concrete slab roof. The drawings show the decorative clay tile is anchored to the brick with 3/8-in. diameter rods spaced at 3 ft. o.c. At the top of the clay tile, the rods anchor a 3/8 in. x 3-in. steel strap that is continuous. Anchorage of the concrete slab roof diaphragm to the masonry walls is facilitated by 3/8-in. diameter by 24-in. long rods at 8-in. o.c.

The drawings seem to indicate that the steel trusses are anchored to the walls at their bearing points, however, the connection detail is not specifically called out. The height-to-thickness ratio of the walls of this building is about 13, which is reasonable for a non-reinforced masonry building. With these considerations, we rate this building as having a moderate vulnerability to damage in a Class I seismic event. Based on our preliminary findings, we believe that the building could sustain some damage in a Class I seismic event, though the structural integrity of the building would be maintained. We recommend that

a more in-depth investigation and analysis of the building structure be performed to more accurately determine the seismic risk.

The floor of the chlorination facility that was added is a reinforced concrete beam and slab structure supported on pipe columns. The walls between these facilities are constructed of 8-in. pumice block with a bond beam and reinforcing at the ceiling connection, approximately 10 ft. 6 in. above the floor slab. The chlorination facility structure is judged to have a low vulnerability to damage.

11.16.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This equipment is located near the northwest corner of the Gate House building. The chlorinator cabinet is framed out of lightweight plastic and is anchored in all four corners. Most of the piping connected to the equipment is plastic, which has some flexibility for movement. The steel piping, however, is not that flexible and could rupture during an earthquake. The reliability of this equipment could be improved by providing flexible couplings on the piping. This equipment has a low vulnerability to seismic damage.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method to prevent this potential hazard is to secure the tanks with a strap around their center. Alternatively, removable reinforced bollards can be installed at the ends of the tank rack, along with tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

11.16.4 Summary and Conclusions

a) From this preliminary structural investigation, we have identified a few possible weak links that should be investigated further. After a more in-depth engineering analysis of some of the items listed in the structure section, some remedial strengthening of the building may be required.

b) Most of the chlorination equipment has a low vulnerability to seismic damage. The chlorine tanks should be strapped together as specified above.

c) Table 11.16-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.16-2.

Table 11.16-1

Summary of Preliminary Seismic Evaluations
Green Lake Chlorine Building and Gate House (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Further analysis required
2	Chlorinators	1	1	Consider flexible couplings
3	Chlorine tanks	2	1	Provide hold-down strap
4	Chlorine building addition	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.16-2

Cost Estimate

Location: Green Lake Chlorination Building and Gate House

Date Constructed: 1911 and 1957

Priority: High

Seismic Strengthening Objectives:

1) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing

Upgrade Recommendations:

1) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides

Additional Investigation Required:

1) More detailed evaluation of structure's seismic performance and verification of connection details.

Assumption: Construction Performed In-House

Cost Estimate:

1)	Additional Investigation	\$4,000
2)	Engineering	700
3)	Construction Engineering	100
4)	Construction	1,700
	Subtotal:	6,500
5)	Sales Tax (8.1%)	140
	Total:	\$6,640

Accuracy of Estimate: ±30%

11.17 Highland Park Pump Station

11.17.1 Facility Description

The Highland Park facility, constructed in 1965, is located on 8th Ave. S.W. and S.W. Trenton St. It supplies water to the 575 zone as well as to the Myrtle reservoir and the Beverly Park tank. The below-grade structure consists of cast-in-place 12-in. thick reinforced concrete basement walls on continuous footings. The building measures 20 ft. wide by 48 ft. long. The concrete slab roof is supported by five transverse concrete beams and is 13 ft. above the interior slab.

11.17.2 Structure Evaluations

From the original construction documents, the 12-in. basement perimeter walls are reinforced in each face with #5 bars. All wall reinforcement appears to be doweled adequately to provide acceptable seismic performance. We feel that a Class I type earthquake will impart some visible cracking in a few members and judge this structure as having a low vulnerability to seismic damage.

11.17.3 Equipment and Contents Evaluations

Pumps

Three pumps are contained within the building. These pumps are well anchored and have a low vulnerability to damage.

Electrical Control Panels

Due to the small width of the southern motor control panels on the concrete slab, we recommend that the anchorage of the panels be verified and braced, as necessary, to the south concrete wall at the top and center of the units to prevent overturning during an earthquake.

11.17.4 Summary and Conclusions

a) From this preliminary structural investigation, the structure will perform well in a Class I earthquake with minor damage

b) The anchorage of the motor control equipment to the concrete floor should be verified. If inadequate, the equipment should be anchored to the wall for stability and prevention of conduit rupture.

c) Table 11.17-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.17-2.

Table 11.17-1

Summary of Preliminary Seismic Evaluations
Highland Park Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	Minor cracking
2	Pumps	1	1	Well anchored
3	Electric control panel	3	2	Verify anchorage/provide brace

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.17-2

Cost Estimate

Location: Highland Park Pump Station

Date Constructed: 1965

Priority: Medium

Seismic Strengthening Objectives:

1) Prevent motor control panels from overturning

Upgrade Recommendations:

1) Verify/anchor control panels

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: +30%

11.18 Interbay Pump Station

11.18.1 Facility Description

This facility was constructed in 1911 and is located on the N.E. corner of 23rd Ave. W. and Dravus St. It supplies water to the Magnolia Bluff tank (470 zone) and the Magnolia reservoir. The building is a single-story brick and mortar type structure measuring 26 ft. wide by 40 ft. long with the roof ridge nearly 24 ft. above the finished slab on grade. The roof diaphragm connects to the perimeter brick walls at about 15 ft. above the slab. Roof loads are transferred to the three wood trusses and transverse walls by 3 x 6-in. T & G parallel decking running longitudinally. The trusses are about 10 ft. o.c. and the transverse brick walls are built to match the roof pitch.

11.18.2 Structure Evaluations

Windows in all four exterior brick walls have been filled in with brick and mortar, thus increasing the ability of the exterior walls to carry in-plane loads. The 13-in. thick exterior brick walls are founded on a continuous concrete foundation which has no observed evidence of settlement. There is evidence of settlement visible in the concrete slab on grade near the base of the chimney, which is located at about the center of one of the longitudinal walls.

The roof as it was constructed does not provide for the type of diaphragm action required by the 1985 Uniform Building Code. Roof loads are transferred to the three wood trusses and transverse walls by 3 x 6-in. T & G parallel decking running longitudinally. With massive brick walls such as these, a positive direct connection to the roof diaphragm at frequent intervals is required to distribute the inertial force of the wall into the diaphragm during a seismic event. Although the longitudinal walls had adequate steel straps to the diaphragm, the transverse gable end walls did not. Since this connection is not provided to the transverse walls, they may lose their out-of-plane support, resulting in potential structural failure. With old, unreinforced masonry construction such as this, it is likely that the walls of this structure could suffer major structural damage from a Class I-type earthquake.

11.18.3 Equipment and Contents Evaluations

Pump and Motor Assembly

The pump and motor systems are anchored to a low concrete pedestal with six 1 in. diameter anchor bolts. The piping that is connected to the pump is 12-in. in diameter. This equipment is well-anchored and has a low vulnerability to earthquake effects.

Table 11.18-2

Cost Estimate

Location: Interbay Pump Station

Date Constructed: 1911

Priority: Medium

Seismic Strengthening Objectives:

1) Anchor transverse walls to roof diaphragm
2) Strengthen the roof diaphragm
3) Vertically and horizontally reinforce the exterior walls

Upgrade Recommendations:

1) Anchor transverse walls to roof with steel straps 4-ft. o.c.
2) Remove roofing and add 3/8-in. plywood to existing sheathing
3) Install steel reinforcement to inside of exterior walls and apply gunite

Assumption: Construction Performed Outside

Cost Estimate:

1)	Engineering	\$ 21,900
2)	Construction Engineering	8,900
3)	Construction	40,000
	Subtotal:	70,800
4)	Sales Tax (8.1%)	3,200
	Total:	\$74,000

Accuracy of Estimate: ±35%

Electrical Control Panel

This panel is constructed out of sheet metal and measures 8 ft long, 3.5 ft wide, and 7.5 ft tall. A 5 in. conduit is connected to the control panel at the top. The dimensions are such that the panel is not likely to overturn when subjected to a Class I-type earthquake. This equipment has a low vulnerability to earthquake damage.

11.18.4 Summary and Conclusions

a) Since this facility was constructed in the early 1900's, the main concern with the structure is that the heavy masonry walls may loose their support at the top due to lack of an effective roof diaphragm connection. If this separation occurs, significant damage will occur. Therefore, this building is judged to be highly vulnerable to a Class I-type earthquake.

b) The equipment in this facility appears to be relatively rugged and stable. This equipment has a low vulnerability to earthquake damage.

c) Table 11.18-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.18-2.

Table 11.18-1

Summary of Preliminary Seismic Evaluations
Interbay Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	3	2	Poor wall-to-roof connection
2	Pumps	1	1	Well anchored
3	Electric control panels	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.18-2

Cost Estimate

Location: Interbay Pump Station

Date Constructed: 1911

Priority: Medium

Seismic Strengthening Objectives:

1) Anchor transverse walls to roof diaphragm
2) Strengthen the roof diaphragm
3) Vertically and horizontally reinforce the exterior walls

Upgrade Recommendations:

1) Anchor transverse walls to roof with steel straps 4-ft. o.c.
2) Remove roofing and add 3/8-in. plywood to existing sheathing
3) Install steel reinforcement to inside of exterior walls and apply gunite

Assumption: Construction Performed Outside

Cost Estimate:

1)	Engineering	$ 21,900
2)	Construction Engineering	8,900
3)	Construction	40,000
	Subtotal:	70,800
4)	Sales Tax (8.1%)	3,200
	Total:	$74,000

Accuracy of Estimate: ±35%

11.19 Lake Forest Chlorination Building

11.19.1 Facility Description

This facility, located at the northeast corner of 45th Avenue N.E. and N.E. 195th Street, serves the Lake Forest Park reservoir. The building, constructed in 1963, is a single story brick and mortar structure measuring 22 ft. by 27 ft. The roof is comprised of 4 x 6-in. T & G decking, with 3-1/8 x 12-in. glu-lam roof beams resting on 12 x 12-in. pumice block pilasters. The exterior walls are constructed of 4-in. pumice block and 4-in. brick veneer.

11.19.2 Structure Evaluations

Roof loads are transferred through the 4 x 6-in. T & G decking to the three 3-1/8 x 12-in. roof beams supported on the 12 x 12-in. pumice block pilasters. It appears from the construction drawings that the exterior walls are constructed without reinforcing. The brick veneer is apparently tied to the pumice block. According to the construction drawings, the exterior bearing walls have very little, if any, vertical and horizontal reinforcing. The three glu-lam beams that support the roof decking are well anchored to the pilasters with anchor bolts. The construction drawings did not detail the block wall-to-roof connections; therefore, it must be assumed that positive anchorage to the diaphragm as required by the Uniform Building Code is not provided. The interior block walls are constructed of 8-in. pumice block with only one horizontal bond beam with two #5 bars tied to the pilasters. One large crack was noticed in the chlorine tank room over the double door opening. This was the only substantial crack found.

There are several interior walls that serve to break up and lower diaphragm stresses. The nine pumice block pilasters assist in the lateral stability of the structure. However, in using the ATC-14 rapid evaluation checklist, several weaknesses are apparent, due mainly to absent vertical reinforcing. With these considerations, we rate this structure as having a moderate vulnerability to a Class I earthquake and recommend that additional analysis be completed to predict more accurately the performance of the building. Based on our preliminary findings, some cracking and spalling of the walls is expected; however, the structure should be able to remain in service.

11.19.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This equipment is located in the north half of the building. The chlorinator cabinet is framed out of light weight plastic and is anchored in all four corners. Although most of the piping and

conduits are not flexible, the chlorine piping has some flexibility to allow movement. The reliability of this equipment could be increased by providing flexible piping at least to the upper portion of the units. We find this equipment to have a low vulnerability to damage.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method to prevent this potential hazard is to secure the tanks with a strap around their center. Alternatively, removable reinforced bollards can be installed at the ends of the tank rack, along with tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

Chlorine Piping

Much of the chlorine piping that is connected to the equipment doesn't have flexible couplings. These could be installed as a preventative measure.

Miscellaneous Equipment

The two graph recorders and telemetry equipment attached to the block wall in the tank room are well anchored and have a low vulnerability to damage.

11.19.4 Summary and Conclusions

a) The structure has some seismic weaknesses, mostly due to inadequate reinforcing. We recommend that a more detailed investigation of this structure be conducted in order to verify the adequacy of the wall-to-wall connection. Based on our preliminary findings, however, the structure is judged to have low to moderate vulnerability and remedial strengthening does not appear to be warranted.

b) Most equipment in this facility has a low vulnerability to seismic damage. The chlorine tanks need to be secured to prevent them from sliding off their racks. This can be accomplished by strapping the tanks or installing removable bollards at the ends of the tank rack and tube steel rail headers along both sides of the racks.

c) Table 11.19-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.19-2.

Table 11.19-1

Summary of Preliminary Seismic Evaluations
Lake Forest Park Chlorination Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Additional review of wall-to-roof connection required
2	Pumps	1	1	Well supported
3	Chlorine tanks	2	1	Provide hold-down strap
4	Chlorinator and analyzer	1	1	Consider flexible couplings

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.19-2

Cost Estimate

Location: Lake Forest Park Chlorination Building

Date Constructed: 1963

Priority: High

Seismic Strengthening Objectives:

1) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing

Upgrade Recommendations:

1) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides

Additional Investigation Required:

1) Verify block wall-to-roof connections
2) Evaluate effect that lack of vertical reinforcing has on seismic performance of structure

Assumption: Construction Performed In-House

Cost Estimate:

1)	Additional Investigation	$3,000
2)	Engineering	700
3)	Construction Engineering	100
4)	Construction	1,700
	Subtotal:	5,500
5)	Sales Tax (8.1%)	140
	Total:	$5,640

Accuracy of Estimate: ±30%

11.20 Lake Hills Pump Station

11.20.1 Facility Description

The Lake Hills facility, constructed in 1973, is located on 140th Pl. S.E. and S.E. 16th St. It supplies water to the 493 and 760 zones as well as supplying water to the Eastside reservoir. The below grade structure consists of cast-in-place, 12-in. thick reinforced concrete basement walls on continuous footings. The building measures 22 ft. wide by 33 ft. long. The concrete slab roof is supported by three transverse concrete beams and is 13 ft. above the interior slab.

11.20.2 Structure Evaluations

From the original construction documents, the 12-in. basement perimeter walls are reinforced in each face with #5 bars according to minimum requirements of the current ACI code. All wall reinforcement appears to be doweled adequately to provide acceptable seismic performance. In a Class I-type earthquake, some visible cracking will occur in selected members. This structure is judged as having a low vulnerability to seismic damage.

11.20.3 Equipment and Contents Evaluations

Pumps

Two pumps are contained within the building. These pumps are well anchored and have a low vulnerability to damage.

Electrical Control Panels

Due to a large height-to-width ratio, the eastern motor control panels on the concrete slab are susceptible to overturning. The anchorage should be verified. If inadequate, it is recommended that the panels be braced to the south concrete wall at the top and center of the units to prevent overturning during an earthquake. The telemetry equipment is well anchored to the concrete walls.

11.20.4 Summary and Conclusions

a) From this preliminary structural investigation, the Lake Hills facility has a low vulnerability to a Class I seismic event.

b) Anchorage of the motor control equipment to the floor slab should be verified. If inadequate, the equipment should be anchored to the wall for stability and to prevent conduit rupture. The telemetry equipment is well supported and has a low vulnerability

c) Table 11.20-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.20-2.

Table 11.20-1

Summary of Preliminary Seismic Evaluations
Lake Hills Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panel	2	1	Verify anchorage and provide brace, as necessary

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.20-2

Cost Estimate

Location: Lake Hills Pump Station

Date Constructed: 1973

Priority: High

Seismic Strengthening Objectives:

1) Prevent control panels from overturning

Upgrade Recommendations:

1) Verify/anchor control panels

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: +30%

11.21 Lincoln Park Pump Station and Gate House

The Lincoln Park facility is located at 10th Avenue and Olive Street, south of the Lincoln Park Reservoir. The original Gate House was built in 1900 and supplies water to the 520 zone and the downtown area. The approximate shape of the structure is a rectangle measuring 28 ft. by 41 ft. with two half-circles on each end with 14 ft. radii. The total length of the structure is about 70 ft.. The upper structure is founded on a massive, unreinforced concrete base that extends to about 30 ft. below grade at the west end of the building. All of the original window openings have been filled in with block.

11.21.1 Structure Evaluations

The original construction drawings show 16-in. thick unreinforced concrete walls that extend 20 ft. from the ground floor to the concrete roof slab. The concrete roof slab is approximately 6-in. thick and is supported by steel "I" beams placed within the concrete roof section. The connection of the concrete roof slab to the exterior concrete walls appears to be a keyed connection. Thus, the transfer of the roof diaphragm shear to the walls will be accomplished by shear friction. During the inspection, many wall cracks were noticed. This is probably due to the lack of steel reinforcement within the walls. Since the concrete foundation is so massive, we find this portion of the building to have a low vulnerability to seismic damage. In our opinion, this building has a moderate vulnerability to seismic damage, though structural integrity should be sustained. We recommend that a more in-depth analysis be completed to more accurately assess the seismic risk involved. Remedial strengthening is probably not warranted, based on the preliminary findings.

11.21.2 Equipment and Evaluations

Miscellaneous Equipment

Most of the equipment consists of wall mounted electrical panels, telemetry and automatic valve controls, and chart recorders. This equipment has a low vulnerability to seismic damage. The hydraulic pump is well anchored to its concrete foundation.

11.21.3 Summary and Conclusions

a) From this preliminary structural investigation, a few possible weak links were identified that should be investigated further. It is recommended that a more in-depth engineering analysis of the structure be performed.

b) The equipment within the building has a low vulnerability to seismic damage.

c) Table 11.21-1 summarizes the results of the seismic evaluations.

Table 11.21-1

Summary of Preliminary Seismic Evaluations
Lincoln Park Pump Station and Gate House (High Priority)

Item	Description	Class I	Class II	Comment
1	Gate house	2	1	More analysis required
2	Equipment	1	1	Well supported

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.21-2

Cost Estimate

Location: Lincoln Park Pump Station and Gate House

Date Constructed: 1900

Priority: High

Additional Investigation Required:

1) Verification of block wall-to-roof connections
2) Detailed analysis of seismic performance of structure

Cost Estimate:

1)	Engineering	$5,000
	Total:	$5,000

Accuracy of Estimate: ±20%

11.22 Magnolia Manor Chlorination Building

11.22.1 Facility Descriptions

This facility, situated at the north end of Magnolia Manor reservoir, is located near 27th Avenue W. and W. Bertona Street. The building was constructed in 1958. It is a single story block and mortar structure measuring 16 ft. by 18 ft. The roof is comprised of 2 x 12-in. T & G decking resting on three 4 x 12-in. roof beams. The exterior walls are constructed of 8-in. pumice block and 4-in. brick veneer.

11.22.2 Structure Evaluations

Roof loads are transferred through the 2 x 6-in. T & G decking to three 4 x 12-in. roof beams, two of which bear on the longitudinal walls, the other is a ridge beam with a post supported by each transverse wall. The construction drawings of the exterior walls show that the 4-in. brick veneer is attached to the 8-in. pumice block with "Z" bar ties. The block walls are only reinforced horizontally, with "K - web" every third course. A bond beam with two #5 bars is continuous around the perimeter of the building, approximately 8 ft. above the finished floor. The block walls are attached to the perimeter footing with #5 bar stubs at 4 ft. o.c. The transverse walls are built up above the bond beam with 2 x 10-in. members at 16-in. o.c. to create a roof pitch.

The building was found to be in very good condition. No wall cracking or settlement was observed during the inspection. A few structural weaknesses were identified from a review of the construction drawings: 1) no vertical reinforcement was specified as required by today's codes; 2) the block walls at the transverse elevations were not positively connected to the roof diaphragm, as they do not extend to the roof diaphragm. Although in many buildings these weaknesses could produce a severe seismic risk, this particular building is considered to be only a low to moderate risk since its size is relatively small, and chord steel is provided at the top of the block walls.

In our opinion, a Class I seismic event will cause this building to sustain some damage such as wall cracking and yielding; however, the structural integrity of the building will be sustained. Thus, we believe that the structure has a low vulnerability to seismic damage. The damage that is expected to occur will be minor, and the facility should remain operable.

11.22.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

The chlorine equipment is located in the west half of the building. The chlorinator cabinet is framed out of light-weight plastic and is anchored in all four corners. All of the piping and conduits attached to the units were found to have flexible lines. The chlorine supply line is coiled 3/8-in. steel tubing. This tubing has some flexibility which should be sufficient to prevent rupture. This equipment has a low vulnerability to damage.

150 lb. Chlorine Cylinders

The chlorine cylinders are anchored to the block walls with chains, both when stored and when they are in use on the scales. Thus secured, the tanks have a low vulnerability to damage.

Chlorine Piping

Much of the chlorine piping that is connected to the equipment has flexible couplings. The PVC piping also provides the desired flexibility.

Miscellaneous Equipment

One recorder is attached to the block wall, along with several electrical panels. All are well anchored and have a low vulnerability to damage. The telemetry equipment and chlorine detector unit are also attached to the block walls and are well anchored. These units and the others attached to the walls have a low vulnerability.

11.22.4 Summary and Conclusions

a) The structure has a few weaknesses, mostly due to inadequate reinforcing and anchorage of some block walls. However, due to the small size of the structure, it is judged to have low vulnerability.

b) The equipment in the facility is well anchored and has a low vulnerability to seismic damage.

c) Table 11.22-1 summarizes the results of the seismic evaluations.

Table 11.22-1

Summary of Preliminary Seismic Evaluations
Magnolia Manor Chlorination Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1-2	1	Lack of wall support at top
2	Chlorine Cylinders	1	1	Well anchored
3	Chlorinator and Analyzer	1	1	O.K.
4	Piping	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.23 Maple Leaf Chlorination Building

11.23.1 Facility Descriptions

This facility, located near the Maple Leaf pump station at 12th Avenue N.E. and N.E. 82nd Street, serves the Maple Leaf reservoir. The building was constructed in 1957. It is a single story brick and mortar structure measuring 16 ft. by 27 ft. The roof is comprised of 2 x 6-in. T & G decking supported by three 4 x 12-in. roof beams.

11.23.2 Structure Evaluations

According to the construction drawings, the exterior bearing walls consist of 8-in. c.m.u. with a 4-in. brick veneer facing. The walls have horizontal bond beams with two continuous #5 bars and K-web reinforcing at every third course. No vertical reinforcing was called for in the drawings; however, the walls are doweled with a #5 bar at 48-in. o.c. The 4 x 12-in. roof beams rest on the longitudinal walls and the transverse walls have a short 2 x 10-in. stud wall extending to the roof diaphragm. The walls are not positively connected to the roof diaphragm as required by the current Uniform Building Code.

At the top of the c.m.u. walls, the bond beam with two continuous #5 bars assists in the lateral stability of the structure; however, using the ATC-14 rapid evaluation checklist several weaknesses are noted due mainly to absent vertical reinforcing. The structure also has a center partition wall that assists in the seismic resistance of the building. Given these considerations, we rate this structure as having a low vulnerability to a Class I earthquake, and do not recommend that remedial strengthening be undertaken.

11.23.3 Equipment and Contents Evaluations

Chlorinators

The chlorinators are located in the west half of the building. The cabinet is framed out of light weight plastic and is anchored in all four corners. This equipment has a low vulnerability to damage, however its seismic resistance could be improved by using flexible couplings on the piping to these units.

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method to prevent this potential hazard is to secure the tanks with a strap around their center. Alternatively, removable reinforced bollards can be installed at the ends of the tank rack, along with tube

steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

11.23.4 Summary and Conclusions

a) Although the structure has a few weak links, the building is small and the center partition wall helps to relieve diaphragm stress. Wall continuity is provided at the top of the c.m.u. walls by a bond beam. Given these considerations and without a detailed analysis, some damage may occur in a Class I earthquake; however, the overall structural integrity will be sustained.

b) The equipment in this facility generally has a low vulnerability to seismic damage. The chlorine tanks need to be secured to prevent them from sliding off their racks. This can be accomplished by strapping the tanks or installing removable bollards at the ends of the tank rack and tube steel rail headers along both sides of the racks. Seismic resistance of the chlorinators could be improved by providing flexible piping couplings.

c) Table 11.23-1 summarizes the results of the seismic evaluations. An estimate of remedial upgrade costs is presented in Table 11.23-2.

Table 11.23-1

Summary of Preliminary Seismic Evaluations
Maple Leaf Chlorination Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Chlorine tanks	2	1	Provide hold-down strap
3	Chlorinator and analyzer	1	1	Consider flexible couplings
4	Electric control panel	1	1	Well anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.23-2

Cost Estimate

Location: Maple Leaf Chlorination Facility

Date Constructed: 1957

Priority: High

Seismic Strengthening Objectives:

1) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing

Upgrade Recommendations:

1) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 700
2)	Construction Engineering	100
3)	Construction	1,700
	Subtotal:	2,500
4)	Sales Tax (8.1%)	140
	Total:	$ 2,640

Accuracy of Estimate: ±25%

11.24 Maple Leaf Pump Station and Gate House

11.24.1 Facility Description

The Maple Leaf facilities are located at 12th Avenue N.E. and N.E. 82nd Street, south of the Maple Leaf Reservoir. The original Gate House was built in 1911 and the adjacent underground pump station facility was added in 1984. The gate house measures 30 ft. by 38 ft. and is very similar to the Green Lake Gate House. The walls consist of 13 in. brick masonry extending to about 14 ft. 5 in. above the interior intermediate concrete floor slab. The roof is framed with pitched steel trusses at about 8 ft. o.c. with intermediate steel "I" sections placed above the trusses. A concrete roof slab is placed over the trusses that varies in thickness from 3-1/2 in. to 6 in. and is reinforced with a wire mesh.

The structure has a basement which is approximately 13 ft. from the slab on grade to the top of the intermediate floor slab. The intermediate floor slab consists of a 4-1/2 in. reinforced concrete slab supported on steel "I" beams and steel columns. The original intermediate floor slab area covered all but a 16 ft. by 10 ft. area and a 15 ft. by 15 ft. area. When the new pump station was added adjacent to the gate house, a 15 ft. by 19 ft. section of the original intermediate floor slab at the northwest corner of the gate house was renovated. The foundation at the ground level measures 18-in. thick and widens to 24 in. at the base with a 3-ft. wide continuous perimeter footing. All of the window openings have been filled in with block.

The new pump station is located underground and to the northwest of the existing gate house. The structure measures 36 ft. by 31 ft. and consists of reinforced concrete walls on a mat foundation. In general, the structure and the equipment within the pump station follows current practices with respect to earthquake design.

11.24.2 Structure Evaluations

According to the original construction drawings, the walls of the Gate House are 13 in. thick, unreinforced brick extending to about 12 ft. above the perimeter foundation. On top of this brick wall, the decorative clay tile extends 3 ft. to meet the concrete slab roof. The drawings show the decorative clay tile is anchored to the brick with 3/8-in. diameter rods spaced at 3-ft. o.c. At the top of the clay tile, the rods anchor a 3/8 in. x 3 in. steel strap that is continuous. Anchorage of the concrete roof slab diaphragm to the masonry walls is facilitated by 3/8-in. diameter by 24-in. long rods at 8-in. o.c. It is unknown if these rods are grouted into the top course of the tile. If they are, they would provide a good structural connection.

The drawings indicate that the steel trusses are anchored to the walls at the bearing point; however, details of this connection are not specifically called out. The height-to-thickness ratio of the walls of this building is about 13, which is reasonable for a non-reinforced masonry building. With these considerations, this building is rated as having a moderate vulnerability to damage in a Class I seismic event. A more in-depth investigation and analysis of the building is recommended to assess more accurately the seismic risk involved; however, at this time it is envisioned that remedial strengthening will not be required.

11.24.3 Equipment and Contents Evaluations

New Mechanical Units

The new mechanical equipment installed with the new pump station is attached to the existing gate house steel trusses with connections designed to allow movement in an earthquake. This equipment has a low vulnerability to seismic damage.

Miscellaneous Equipment

The valves within the gate house have a low vulnerability to damage. There is little other equipment in the gate house.

11.24.4 Summary and Conclusions

a) From this preliminary structural investigation, a few possible weak links have been identified that should be investigated further. After a more in depth engineering analysis of the structure, it is envisioned that remedial strengthening of the gate house building will not be required. The new pump station building has a low vulnerability to damage and no further investigation is required.

b) The equipment within the pump station and gate house buildings has a low vulnerability to seismic damage.

c) Table 11.24-1 summarizes the results of the seismic evaluations.

Table 11.24-1

Summary of Preliminary Seismic Evaluations
Maple Leaf Pump Station and Gate House (High Priority)

Item	Description	Class I	Class II	Comment
1	Gate house	2	1	More analysis required
2	Equipment	1	1	Well anchored
3	Pump station	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.24-2

Cost Estimate

Location: Maple Leaf Pump Station and Gate House

Date Constructed: 1911 (Gate House) and 1984 (Pump Station)

Priority: High

Additional Investigation Required:

1) More detailed evaluation of Gate House structure to verify connection details and seismic performance.

Cost Estimate:

1)	Engineering	$3,600
	Total:	$3,600

Accuracy of Estimate: ±20%

11.25 Maplewood Pump Station

11.25.1 Facility Description

This facility is located at the end of Union Avenue S.E. and S.E. 128th Street in Renton. It is used as a booster pump station to the Bellevue and Mercer Island areas. The building, constructed in 1963, is a single-story block and mortar structure measuring 25 ft. by 40 ft. It's lower basement area is constructed of concrete retaining walls around the perimeter which extend up to ground elevation. The exterior walls are composed of 8-in. block that extend to the roof diaphragm above the concrete retaining walls. Interior 12-in. square concrete columns rise from the floor slab to the roof beams.

11.25.2 Structure Evaluations

Roof loads are transferred through 3 x 6-in. T & G decking to two 2 in. x 10 in. built-up wood beams at approximately 9-ft. o.c. The roof ridge is located at about 9 ft. 6 in. from one of the longitudinal walls and is supported by three 2 x 8-in. built-up beams spanning interior concrete columns. The building also has an intermediate concrete floor slab with dimensions of about 11 ft. by 25 ft., which has a low vulnerability to seismic damage.

According to the construction drawings, the exterior walls consist of 8-in. hollow c.m.u. block with no detail of vertical reinforcing. Horizontal K-web joint reinforcing is provided at each third course. The roof framing is such that the masonry walls are essentially non-bearing. No positive direct connection of the roof diaphragm to the masonry walls exists. Even though the walls are relatively light, it is possible that they could fail out of plane, especially at the transverse gable ends. There are only two door openings in the building; therefore, the wall is not highly susceptible to failure due to in plane loads.

The top of the c.m.u. walls has a bond beam with two continuous #5 bars which border the perimeter of the concrete columns. If the bond beam was securely connected to the concrete columns, this would assist in the lateral stability of the structure. Inspection using the ATC-14 rapid evaluation check list showed several weakness due mainly to the lack of reinforcing and positive anchorage of the walls to the roof diaphragm. The lower portion of the building, consisting of concrete walls and an intermediate floor slab, appears to be adequately detailed to resist seismic forces

With these considerations, we rate this structure as having a moderate vulnerability to a Class I earthquake.

11.25.3 Equipment and Contents Evaluations

Electrical Control Equipment

The electrical motor control panels on the upper concrete floor slab appear to be anchored to the slab; however, this should be verified. If they are properly anchored, their vulnerability to seismic damage is low. All of the other electrical panels attached to the walls at the upper slab level are well anchored and appear to have low vulnerability. The air conditioning unit located on the lower floor slab has a low vulnerability to damage. Another electrical panel attached to the lower concrete wall appears to be well anchored.

Pump and Telemetry Equipment

The pump and telemetry equipment is situated on the lower floor slab. This equipment is securely anchored and has a low vulnerability.

11.25.4 Summary and Conclusions

a) Although the structure has a few weaknesses, the building is small and only one story above the ground, which reduces its vulnerability to seismic damage. Wall continuity at its top could be provided by connecting the top bond beam to each concrete column. This could be accomplished by bolting an angle to one face of the concrete column with two concrete expansion anchors and through bolting the bond beam to the angle. Thus the block wall could be made to span horizontally between the concrete columns, assuming that the concrete columns have adequate capacity.

   A more detailed analysis should be completed to determine the strength of the T & G roof diaphragm, and to assess if steel straps from the diaphragm to the block wall are necessary for the longitudinal walls, as they are with the transverse gable end walls. The existing roof diaphragm is most likely adequate, since the roof beams tend to break the diaphragm into sub-diaphragms.

b) The electrical control equipment appears to be anchored adequately. Verification of anchorage for the motor control units should be performed. The pump and telemetry equipment is well anchored.

c) Table 11.25-1 summarizes the results of the seismic evaluations. An estimate of remedial upgrade costs is presented in Table 11.25-2.

Table 11.25-1

Summary of Preliminary Seismic Evaluations
Maplewood Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	2	Lack of wall support at top
2	Pumps	1	1	Well anchored
3	Electric Equipment	1	1	Verify anchorage

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.25-2

Cost Estimate

Location: Maplewood Pump Station

Date Constructed: 1963

Priority: High

Seismic Strengthening Objectives:

1) Improve lateral wall continuity
2) Evaluate the adequacy of the roof diaphragm
3) Verify anchorage of motor control panel

Upgrade Recommendations:

1) Bolt the bond beam to the columns with straps and brackets
2) Verify anchorage of motor control panel and upgrade, as required

Additional Investigation Required:

1) Investigate the strength of the T & G diaphragm

Assumption: Construction Performed Outside (Except for anchorage of control panel)

Cost Estimate:

1)	Additional Investigation	$3,000
2)	Engineering	10,500
3)	Construction Engineering	6,300
4)	Construction	15,000
	Subtotal:	34,800
5)	Sales Tax (8.1%)	1,200
	Total:	$36,000

Accuracy of Estimate: ±35%

11.26 S.W. Myrtle Chlorination Building

11.26.1 Facility Description

This facility, situated at the north end of the S.W. Myrtle reservoir, is located near 36th Avenue S.W. and S.W. Willow Street. The building was constructed in 1958. It is a single-story block and mortar structure measuring 16 ft. by 23 ft. The roof is comprised of 2 x 6-in. T & G decking.

11.26.2 Structure Evaluations

Roof loads are transferred through the 2 x 6-in. T & G decking to three 4 x 12-in. roof beams, two of which bear on the longitudinal walls continuously and the other is a ridge beam with a post support on each transverse wall and the center bearing wall.

The construction drawings show exterior walls constructed of 8-in. pumice block and a 4-in. brick veneer attached to the block walls with "Z" bar ties. The block walls are only reinforced horizontally with "K - web" every third course. A bond beam with two #5 bars approximately 8 ft. above the finished floor is continuous around the perimeter of the building. The block walls are attached to the perimeter footing with #5 bar stubs at 4-ft. o.c. The transverse walls are built up above the bond beam with 2 in. x 10 in. at 16-in. o.c. to create a roof pitch.

The building was found to be in very good shape. No wall cracking or settlement was noticed during the inspection. A few weaknesses were found in this building when the construction drawings were reviewed: 1) no vertical reinforcement was detailed, as is required by current codes; 2) the block walls at the transverse elevations were not positively connected to the roof diaphragm, since they did not extend to the roof diaphragm. Although in many buildings these weaknesses could mean a severe seismic risk, this particular building is considered to have only a low to moderate risk since its size is relatively small, an interior bearing wall partition exists, and chord steel is provided at the top of the block walls.

We conclude that in a Class I seismic event, this building will sustain some damage such as wall cracking and yielding; however, the structural integrity of the building will be sustained. Thus, in our opinion, the structure has a low vulnerability to seismic damage, and the facility should remain operable.

11.26.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This equipment is located in the north half of the building. The chlorinator cabinet is framed out of light weight plastic and is anchored in all four corners. All of the piping and conduits attached to the units were found to have flexible lines, or the piping was made of PVC which had some flexibility. The chlorine steel supply line has some flexibility in its coiled 3/8 in. steel tubing. We find this equipment to have a low vulnerability to damage.

150 lb. Chlorine Cylinders

The chlorine cylinders are anchored to the block walls with chains. The cylinders when in use rest on the scale and are also chained to the block walls. These cylinders are considered as having a low vulnerability to damage.

Chlorine Piping

Much of the chlorine piping that is connected to the equipment has flexible couplings adjacent to the equipment. The PVC piping is especially earthquake-resistant since it is flexible.

Miscellaneous Equipment

Several electrical panels are attached to the block walls. All are well anchored and have a low vulnerability to damage. The telemetry equipment and chlorine detector unit are also attached to the block walls and are well anchored. These units and the others attached to the walls have a low vulnerability to seismic damage.

11.26.4 Summary and Conclusions

a) The structure has a few weaknesses due mostly to inadequate reinforcing and anchorage of some block walls. However, the structure is judged to have low vulnerability due to its small size.

b) The equipment at this facility has a low vulnerability to seismic damage.

c) Table 11.26-1 summarizes the results of the seismic evaluations.

Table 11.26-1

Summary of Preliminary Seismic Evaluations
S.W. Myrtle Chlorination Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Chlorine cylinders	1	1	Adequate
3	Chlorinator and Analyzer	1	1	Flexible couplings provided
4	Piping	1	1	Well supported or flexible

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.27 North City Pump Station

11.27.1 Facility Description

The North City facility, located near 8th Ave. N.E. and N.E. 185th St., was constructed in 1968 and supplies water to the 580 zone as well as to the Richmond Highland tanks. The below grade structure consists of cast-in-place 12-in. thick reinforced concrete basement walls on continuous footings. The building measures 20-ft. wide by 48-ft. long. The concrete slab roof is supported by five (5) transverse concrete beams and is 13 ft. above the interior slab.

11.27.2 Structure Evaluations

From the original construction documents, the 12 in. basement perimeter walls are reinforced in each face with #5 bars. All wall reinforcement appears to be doweled adequately to provide acceptable seismic performance. A Class I-type earthquake may impart some visible cracking in a few members, however this structure is judged as having a low vulnerability to seismic damage.

11.27.3 Equipment and Contents Evaluations

Pumps

The three pumps contained within the building are well anchored and have a low vulnerability to damage.

Electrical Control Panels

Due to their large height-to-width ratio, the southern motor control panels on the concrete slab are susceptible to overturning in a major earthquake. We recommend that the anchorage of the panels to the concrete floor be verified. The units should at least be bolted to the floor with an anchor bolt at each of the four corners. If found inadequate, we recommend that the panels be braced to the south concrete wall at the top and center of the units to prevent overturning.

The air conditioning unit rests on a 12-in. concrete pedestal but is not anchored to the concrete with anchor bolts. Four concrete expansion anchors should be used to secure the unit to the pedestal.

11.27.4 Summary and Conclusions

a) From this preliminary structural investigation, the structure is judged to have low vulnerability.

b) The anchorage of the motor control equipment should be verified, and if necessary, the equipment should be anchored to

the wall for stability and prevention of conduit rupture. The air conditioning unit should be anchored to its concrete pedestal.

c) Table 11.27-1 summarizes the results of the seismic evaluation. An estimate of the remedial upgrade costs is presented in Table 11.27-2.

Table 11.27-1

Summary of Preliminary Seismic Evaluations
North City Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panel	2	1	Verify anchorage; provide brace if necessary
4	A/C unit	3	2	Provide anchor bolts

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.27-2

Cost Estimate

Location: North City Pump Station

Date Constructed: 1968

Priority: Medium

Seismic Strengthening Objectives:

1) Verify motor control equipment anchorage, secure to wall as necessary
2) Secure air conditioner to concrete pedestal

Upgrade Recommendations:

1) Bolt motor control equipment to wall with 4 concrete anchors (if no anchorage is provided)
2) Secure air conditioner to concrete pedestal with 4 bolts

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 200
2)	Construction Engineering	0
3)	Construction	1,000
	Subtotal:	1,200
4)	Sales Tax (8.1%)	80
	Total:	$1,280

Accuracy of Estimate: ±30%

11.28 Northgate Pump Station

11.28.1 Facility Description

The Northgate facility, constructed in 1950, is located on the N.E. corner of 4th Ave. N.E. and N.E. 115th St. This facility is presently not in use. The structure is a cast-in-place reinforced concrete building on continuous footings. The building measures 29-ft. wide by 30-ft. long. The concrete slab roof is supported by one concrete beam and is nearly 10 ft. above the floor slab. The original window openings have been filled in with c.m.u. block. An intermediate floor slab exists above the slab on grade on which the pump equipment is located. No evidence of settling was observed in this building although some minor wall cracks were observed in some of the walls.

11.28.2 Structure Evaluations

From the original construction documents, the 8-in. exterior concrete perimeter walls extend about 4 ft. above the upper interior floor slab, with 6-in. walls continuing to the roof slab. The 8-in. walls are provided with two faces of reinforcing and the 6-in. walls with one. The connection of the walls to the slab diaphragms appear to be adequate to resist seismic forces. Though the walls are not doweled to the continuous footings, a 2 x 4-in. shear key is provided which will probably be sufficient. In conclusion, we estimate that a Class I earthquake will result in some visible cracking or yielding of a few members in the building; however, the structural integrity of the building should not be affected.

11.28.3 Equipment and Contents Evaluations

Pumps

Two pumps are contained within the pump station which are anchored on concrete pedestals. This equipment is considered to have a low vulnerability to earthquake damage.

Electrical Control Panels

The motor control panels appear to have a low vulnerability to seismic damage due to their relatively large width.

11.28.4 Summary and Conclusions

a) For this preliminary investigation it appears that this structure and its equipment will perform well in a Class I earthquake.

b) Table 11.28-1 summarizes the results of the seismic evaluations.

Table 11.28-1

Summary of Preliminary Seismic Evaluations
Northgate Pump Station (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panel	1	1	Stable

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.29 Riverton Hts. Well and Treatment Facility

11.29.1 Facility Description

This facility, located at 24th Avenue S. and S. 148th Street, supplies water to West Seattle Pipeline No. 4 and also serves as a water treatment facility. The building was constructed in 1987. It is a single-story, structural brick and mortar structure measuring 35 ft. by 47 ft. The roof consists of prefabricated trusses at 24-in. o.c. with an attached plywood roof diaphragm. Several interior non-bearing walls serve to separate the treatment rooms from the pump rooms.

11.29.2 Structure Evaluations

According to the construction drawings, the exterior bearing walls consist of 6 x 12-in. structural brick reinforced with #5 vertical bars at 48-in. o.c., and two #5 horizontal bars at 48-in. o.c. The walls also have horizontal joint reinforcing at every fourth course. All of the vertical wall reinforcing is doweled into the concrete perimeter foundation.

The roof trusses bear on a double 2 x 8-in. top plate that is anchored to the top brick bond beam with bolts at 24-in. o.c. The trusses are attached to the wood plate with a Simpson H1, which appears to be a seismically-adequate connection. The exterior walls of the building have only a few penetrations, which suggest that the shear stress in the masonry is not critical with the reinforcing provided. The interior walls within the structure are well anchored and provide additional stability. The interior walls are positively attached to the bottom chord of the trusses with an adequate seismic detail.

With these considerations, we rate this structure as having a low vulnerability to a Class I earthquake.

11.29.3 Equipment and Contents Evaluations

Chemical Tanks

The chemical tanks in the chemical room are set into a concrete saddle and are anchored with straps to the saddle. This equipment has a low vulnerability to seismic damage.

Pumps and Electrical Control Panels

The pumps and piping are very well anchored to large concrete footings and are considered to have low vulnerability to damage. The chlorine, well monitoring and pump control equipment are all well anchored except for the Westinghouse transformer XFMR T-1. This unit should be anchored. In addition, motor control anchorage should be verified.

11.29.4 Summary and Conclusions

a) This structure has recently been designed and constructed. In our opinion, the structure will perform well when subjected to a Class I earthquake.

b) The Westinghouse transformer XFMR T-1 should be anchored.

c) Table 11.29-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.29-2.

Table 11.29-1

Summary of Preliminary Seismic Evaluations
Riverton Heights Well Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Chemical tanks	1	1	Well anchored
3	Pumps	1	1	Well anchored
4	Motor control consoles	2	1	Verify anchorage
5	Transformer XFMR T-1	3	2	Unanchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.29-2

Cost Estimate

Location: Riverton Heights Well & Treatment Facility

Date Constructed: 1987

Priority: High

Seismic Strengthening Objectives:

1) Prevent Westinghouse transformer unit XFMR T-1 from overturning

Upgrade Recommendations:

1) Bolt XFMR T-1 to floor

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: ±30%

11.30 Roosevelt Way Pump Station

11.30.1 Facility Description

The Roosevelt facility was constructed in 1952 and is located on the corner of Roosevelt Way and E. 82nd St. It supplies water to the 520 zone. The structure consists of a cast-in-place reinforced concrete partial basement area and a one-story building above on continuous footings. The building measures approximately 28-ft. wide by 42-ft. long. The building has an irregular plan. The roof diaphragm consists of 2 x 6-in. T & G decking over 2 x 8-in. wood joists at 16-in. o.c. The roof perimeter is then supported on masonry or concrete walls.

11.30.2 Structure Evaluations

From the original construction documents, two types of exterior masonry walls exist. A 20-ft. section of the south elevation wall consists of an 8 in. unreinforced brick wall extending to the roof joists; a 13 ft. portion of the west elevation wall is 12-in. thick unreinforced sandstone. The connection of these walls to the roof diaphragm is inadequate, and produces cross grain tension in the wood plates at the top of the walls. The connection of the roof diaphragm to the exterior concrete walls also appear to be inadequate, and stresses the wood support members across the grain.

In a Class I-type earthquake, it is judged that this structure will not provide acceptable seismic performance. It is expected to have a moderate-to-high vulnerability to seismic damage.

11.30.3 Equipment and Contents Evaluations

Pumps

Three pumps contained within the building are well anchored and have a low vulnerability to damage.

Electrical Control Panels

The anchorage of the motor control panel on the concrete platform should be verified and secured if necessary.

11.30.4 Summary and Conclusions

a) Through this preliminary structural investigation, several weaknesses were found in the seismic resistance of this building. Improvements could be made to improve the seismic resistance of this structure:

1) Remove the existing roofing and supply a new plywood diaphragm to the existing 2 x 6-in. T & G roof. Anchor the 2 x 8-in. roof joists to the masonry walls with steel straps at 4-ft. o.c. This could also be done along the concrete perimeter walls. A method for reinforcing the existing 8-in. brick wall should be investigated.

2) The existing heavy masonry wall needs to be either reinforced with steel mesh and gunite applied to the interior face of the wall or replaced with a new, seismic-resistant wall.

b) The anchorage of the motor control equipment should be verified.

c) Table 11-30-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.30-2.

Table 11.30-1

Summary of Preliminary Seismic Evaluations
Roosevelt Pump Station (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2-3	2	Unreinforced masonry walls; Poor roof diaphragm
2	Pumps	1	1	Well anchored
3	Electric control panel	2	1	Verify anchorage

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.30-2

Cost Estimate

Location: Roosevelt Pump Station

Date Constructed: 1952

Priority: Low

Seismic Strengthening Objectives:

1) Provide direct positive connection of walls to roof diaphragm
2) Strengthen roof diaphragm
3) Reinforce 8-in. brick wall horizontally and vertically
4) Verify anchorage of motor control panel

Upgrade Recommendations:

1) Add plywood diaphragm to existing T&G roof
2) Anchor roof joists to masonry walls using steel straps at 4-ft. o.c.
3) Add internal steel bracing arrangement or steel mesh and gunite reinforcement to 8-in. brick wall
4) Verify anchorage of motor control panel and upgrade, as necessary

Assumption: Construction Performed Outside (Except for anchorage of control panel)

Cost Estimate:

1)	Engineering	$ 21,400
2)	Construction Engineering	10,300
3)	Construction	50,000
	Subtotal:	81,700
4)	Sales Tax (8.1%)	4,100
	Total:	$85,800

Accuracy of Estimate: $\pm$35%

11.31 S.W. Spokane St. Pump Station

11.31.1 Facility Description

This facility was constructed in 1928 and is located on S. W. Spokane St. and 33rd. Ave. S.W. It supplies water to the 316 zone and during summer high demand it boosts water to the 488 zone. The building is a single-story brick and mortar type structure measuring 22-ft. wide by 34-ft. long. Roof loads appear to be transferred to wood trusses and transverse walls by 3 x 6-in. T & G parallel decking running longitudinally.

11.31.2 Structure Evaluations

Windows in all four exterior brick walls have been filled in with brick and mortar type construction, thus increasing the ability of the exterior walls to carry in-plane shear stress. The building consists of exterior brick walls (about 13-in. thick on average) on a continuous concrete foundation and no evidence of settlement of the perimeter foundation was observed; however, water seepage was observed in the basement area.

The construction drawings of the roof details were unavailable, thus it was assumed that the connection of the brick walls to the T & G decking does not conform to the 1985 U.B.C.. Massive brick walls such as these, require a positive direct connection to the roof diaphragm at frequent intervals to distribute the inertial force of the wall into the diaphragm. With old unreinforced masonry construction such as this, it is likely that the walls of this structure could suffer major structural damage from a Class I-type earthquake. Further investigation should be undertaken to assess the strength of the roof diaphragm and the adequacy of the connection of the roof to the walls.

11.31.3 Equipment and Contents Evaluations

Pump and Motor Assembly

Two similar pumps, located in the center of the building, are supported on large concrete piers that extend through the double 2 x 6-in. T & G floor system. This equipment is well anchored and is considered to have a low vulnerability to earthquake effects.

Electrical Control Panel

This panel measures 5.5-ft. long, 2.7-ft wide, and 7.5-ft tall. The dimensions are such that the panel may be likely to overturn when subjected to a Class I-type earthquake. This equipment has a moderate vulnerability to earthquake damage, and anchorage of these units should be verified.

11.31.4 Summary and Conclusions

a) Since this facility was constructed using methods in common practice in 1928, the main concern with the structure is that the heavy masonry walls will loose their support at the top due to lack of a positive connection to the roof diaphragm. Therefore, this building should be considered highly vulnerable to a Class I-type earthquake. Since drawings were not available, we recommend that an additional investigation be undertaken to determine the adequacy of the roof connection to the walls.

b) The equipment in this facility appears to be relatively rugged and stable. This equipment has a low vulnerability to earthquake damage, except for the electrical control panel whose anchorage should be verified and braced at the top if necessary.

c) Table 11.31-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.31-2.

Table 11.31-1

Summary of Preliminary Seismic Evaluations
S.W. Spokane St. Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	3	2	Possible lack of wall-to-roof connection
2	Pumps	1	1	Well anchored
3	Electric control panel	2	1	Verify anchorage/provide brace, if necessary

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.31-2

Cost Estimate

Location: S. W. Spokane St. Pump Station

Date Constructed: 1928

Priority: Medium

Seismic Strengthening Objectives:

1) Determine roof connection to walls and adequacy of diaphragm
2) Strengthen the roof diaphragm
3) Provide positive direct connection of brick walls to roof diaphragm
4) Verify anchorage of electrical control panel
5) Remediate stability of hillside slopes adjacent to pump station

Upgrade Recommendations:

1) Remove roofing and add plywood roof diaphragm
2) Anchor brick walls to roof with 1/4-in. steel plate straps at 4-ft. o.c.
3) Anchor top of electrical control panel to wall with two braces
4) Verify anchorage of electrical control panel and brace to wall, as necessary

Additional Investigation Required:

1) Conduct detailed study of roof-to-wall connection and roof diaphragm
2) Conduct detailed study of hillside slope stability

Assumption: Construction Performed Outside (Except for anchorage of control panel)

Cost Estimate:

1)	Additional Investigation	$ 11,000
2)	Engineering	17,900
3)	Construction Engineering	6,300
4)	Construction	37,000
	Subtotal:	72,200
5)	Sales Tax (8.1%)	3,000
	Total:	$75,200

Accuracy of Estimate: $\pm$35%

11.32 Tess Junction Pump Station

11.32.1 Facility Description

The Tess Junction pump station is located near 132nd Avenue, between 149th Street and 151st Street. The pumping station was installed in 1988 to provide emergency backup when the Tolt facilities are not in service. This pumping station supplies water to the highest zone. The structure enclosing the pump station is actually a steel freight container that measures 19 ft. by 7 ft. and is about 7-ft. high. Inside this unit is a pump and motor anchored to a concrete base. The floor of the pump station is crushed rock with a 1-1/8 in. plywood cover.

11.32.2 Structure Evaluations

This container structure is constructed of steel and has a low vulnerability to seismic damage.

11.32.3 Equipment and Contents Evaluations

Pump and Motor

The pump and motor system is anchored to a concrete base and has a low vulnerability to damage. The connections between the motor control panel and the pump are flexible.

Motor Control Panels

The electrical motor control panels are anchored with bolts through the plywood floor and steel plate. The plywood floor is used as a base for the container structure. These panels have a low vulnerability to seismic damage.

11.32.4 Summary and Conclusions

a) This pump station has a low vulnerability to seismic damage.

b) The equipment within the pump station has a low vulnerability to seismic damage.

c) Table 11.32-1 summarizes the results of the seismic evaluations.

Table 11.32-1

Summary of Preliminary Seismic Evaluations
Tess Junction Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.33 S.W. Trenton Pump Station and Gate House

11.33.1 Facility Description

The S.W. Trenton pump station is located next to the S.W. Trenton chlorine facility at 4th Avenue S.W. and S.W. Trenton Street. This facility, originally constructed in 1933, has turbine-powered pumps that supply water to the 575 zone. The structure is cast-in-place reinforced concrete on continuous footings with a basement about 13 ft. below grade. The roof slab is approximately 11 ft. above the concrete floor slab. The plans available for this building were very sketchy, but it appears that the original building measured 22 ft. by 17 ft. in plan. An addition to the building added another 15 foot by 22 foot section. The entire structure measures 22 ft. by 31 ft. The concrete slab roof in the original portion is supported by a center concrete beam. In the newer portion, the concrete slab roof is supported by two steel beams. The original window openings have been filled in with c.m.u. block.

11.33.2 Structure Evaluations

From the original construction documents, the 8-in. exterior concrete perimeter walls appear to be reinforced with #4 bars at 18-in. o.c. both ways each face, which is adequate reinforcing by today's current ACI code. A preliminary analysis of this structure concludes that some visible cracking or yielding may occur; however, the structure should remain operable. Cracking in the ceiling and the walls were apparent during the inspection. The drawings were not complete enough to make a positive determination of the performance of this structure in a Class I event; however, it is felt that the existing cracking indicates that this structure has undergone past distress, possibly caused by previous seismic events. Thus, we estimate that this structure has a moderate vulnerability to seismic damage in a Class I earthquake. Minor damage such as additional wall cracking will likely occur; however, expenditure for seismic strengthening is probably not warranted.

11.33.3 Equipment and Contents Evaluations

Pumps

The two pumps contained within the pump station rest on concrete pedestals and are anchored with bolts. This equipment has a low vulnerability to earthquake damage.

Electric Control Panels

There are several electric panels that are located within the older portion of the building. The electrical equipment is enclosed in rigid

sheet metal cabinets, mostly attached to the concrete walls. This equipment has a low vulnerability to damage.

11.33.4 Summary and Conclusions

a) From the preliminary investigation of the original construction documents, it was not possible to obtain adequate information on some important design details. However, when the ATC checklist for concrete structures was evaluated, this building rated as having a moderate risk. In our opinion, expenditure for seismic strengthening is not warranted.

b) The equipment contained within the structure has a low vulnerability to seismic damage.

c) Table 11.33-1 summarizes the results of the seismic evaluations.

Table 11.33-1

Summary of Preliminary Seismic Evaluations
S.W. Trenton Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	2	1	Expect wall cracks
2	Pumps	1	1	Well anchored
3	Electric control panel	1	1	Anchored

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

11.34 S.W. Trenton Chlorination Building

11.34.1 Facility Description

This facility, located at the end of 4th Avenue S.W., serves the West Seattle reservoir. The building was constructed in 1957. It is a single story pumice block and mortar structure measuring 16 ft. by 27 ft. The roof is comprised of 3 x 6-in. T & G decking with 3 x 6-in. wood ledgers which are bolted to the pumice bond beam.

11.34.2 Structure Evaluations

According to the construction drawings, the exterior bearing walls consist of 8-in. pumice block with a cement stucco finish. The walls have horizontal bond beams with two continuous #5 bars at the roof diaphragm level. No vertical reinforcing was called for in the drawings; however, the walls are doweled with a #5 bar at 48-in. o.c., and the cells are filled at these locations. There are 6 x 12-in. roof beams along the transverse and center walls which rest on 12 x 12-in. pilasters reinforced with four #6 vertical bars with ties at 12 in. o.c. The roof diaphragm connection along the longitudinal walls consists of a 3 x 6-in. ledger bolted to the block with anchor bolts. Cross grain tension is avoided by steel straps embedded into the roof course bond beam. Thus, the walls are positively connected to the roof diaphragm as required by the current Uniform Building Code.

The top of the pumice walls has a bond beam with two continuous #5 bars which contribute to the lateral stability of the structure; however, a review using the ATC-14 rapid evaluation check list exposed several weaknesses, due mainly to absent vertical reinforcing. The structure also has a center partition wall that assists in the seismic resistance of the building. With these considerations and the fact that the building is small, we rate this structure as having a low vulnerability to a Class I earthquake.

11.34.3 Equipment and Contents Evaluations

Chlorine Tanks

The chlorine tanks are situated on scale pits and are supported on small steel rollers. These tanks can slide off their supports during a major earthquake, causing the chlorine feeder lines to rupture. One method to prevent this potential hazard is to secure the tanks with a strap around their center. Alternatively, removable reinforced bollards can be installed at the ends of the tank rack, along with tube steel rail headers paralleling the tank rack on both sides, as shown in Figure 11.40-1.

Chlorine Piping

Much of the chlorine piping that is connected to the equipment does not have flexible couplings at the equipment connections. Flexible couplings could be installed to further reduce the risk of chlorine leaks.

Chlorinator and Chlorine Analyzer

This equipment is located in the south half of the building. The chlorinator cabinet is framed out of light weight plastic and is anchored in all four corners. Although most of the piping and conduits are not flexible, the chlorine piping has some flexibility which allows some movement. The reliability of this equipment could be increased by providing flexible piping at least to the upper portion of the units. This equipment has a low vulnerability to seismic damage.

Miscellaneous Equipment

The two chart recorder units attached to the block wall in the tank room are well anchored and have a low vulnerability to damage. The telemetry equipment also attached to this wall is well anchored and is considered to have a low vulnerability. The chlorine scale in the tank room is well anchored to the concrete slab and is considered to have a low vulnerability.

11.34.4 Summary and Conclusions

a) The structure has a few weaknesses due mainly to inadequate reinforcing; however, the building is small and the center partition wall serves to relieve diaphragm stress. Wall continuity is provided at the top of the pumice walls by a bond beam. With these considerations, and without a detailed analysis, we estimate that although some damage will occur in a Class I earthquake, the overall structural integrity of the building will be sustained.

b) The chlorine tanks should be secured to prevent the chlorine tank feeder lines from rupturing.

c) Table 11.34-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.34-2.

Table 11.34-1

Summary of Preliminary Seismic Evaluations
S.W. Trenton Chlorination Facility (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Chlorine tanks	2	1	Provide hold-down strap
3	Chlorinator and analyzer	1	1	Consider flexible couplings

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.34-2

Cost Estimate

Location: S. W. Trenton Chlorine Facility

Date Constructed: 1957

Priority: High

Seismic Strengthening Objectives:

1) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing

Upgrade Recommendations:

1) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 700
2)	Construction Engineering	100
3)	Construction	1,700
	Subtotal:	2,500
4)	Sales Tax (8.1%)	140
	Total:	$2,640

Accuracy of Estimate: ±25%

11.35 View Ridge Pump Station

11.35.1 Facility Description

The View Ridge facility is located just west of 55th Ave. N.E. on N.E. 75th St. It was constructed in 1978 and is used to supply water to the 520 and 316 zones. The pump station is a cast-in-place reinforced concrete circular below grade structure on a mat foundation with a diameter of 30 ft. The pump station room is contained in the center of the 150-ft. diameter View Ridge reservoir and the pump room equipment floor slab is approximately 10 ft. below the 10.5 in. reservoir top cover slab. The pump room walls are 12-in. thick concrete walls about 23 ft. high with reinforcing in each face. The pump room floor slab is 13 ft. above the reservoir bottom and is supported on two large concrete floor beams. The roof slab has two large openings that are bounded on two sides by roof beams. No evidence of settling or wall cracking was observed in this structure.

11.35.2 Structure Evaluations

From the original construction documents, the concrete perimeter circumferential walls are 12-in. thick, and are provided with #4 bars at 12-in. o.c. vertical each face and #4 at 10-in. o.c. horizontal each face, which complies with the current ACI code. The connection of the walls to the slab diaphragms is adequate to resist seismic forces. The walls are adequately doweled to the mat foundation. The access openings in the roof diaphragm appear to be adequately reinforced to transfer the lateral load through the opening and eventually into the shear walls. In conclusion, we feel that in a Class I-type earthquake, this structure will experience only minor damage which shouldn't affect the structural integrity of the building.

11.35.3 Equipment and Contents Evaluations

Pumps

The two pumps contained within the pump station are anchored adequately. This equipment is considered to have a low vulnerability to earthquake damage.

Electrical Control Panels

The motor control panels are located near the center of the facility. The telemetry equipment is attached to the motor control unit and exits the unit from the top. All appear to be well supported and have a low vulnerability.

Air Conditioning Unit

The air conditioning unit, located adjacent to the motor control units, rests on a 12-in. thick concrete base and appears to be anchored. If so, it has a low vulnerability to seismic damage. The anchorage should be verified, and concrete expansion anchors provided as necessary.

11.35.4 Summary and Conclusions

a) For this preliminary investigation, it appears that this structure will experience only minor structural damage in a Class I earthquake. In our opinion, no need for remedial strengthening is required. Nonstructural damage will also be minor.

b) The anchorage of the air conditioning unit should be verified and provided with concrete expansion anchors if missing.

c) Table 11.35-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.35-2.

Table 11.35-1

Summary of Preliminary Seismic Evaluations
View Ridge Pump Station (Low Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panels/ telemetry	1	1	Well anchored
4	A/C unit	2	1	Verify/supply anchorage

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.35-2

Cost Estimate

Location: View Ridge Pump Station

Date Constructed: 1978

Priority: Low

Seismic Strengthening Objectives:

1) Prevent air conditioning unit from overturning

Upgrade Recommendations:

1) Verify/anchor air conditioning unit

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: +30%

11.36 Volunteer Park Pump Station

11.36.1 Facility Description

The Volunteer Park facility, built in 1958, is located on the NW corner of 12th Ave. E. and E. Prospect St. It supplies water to the 520 zone as well as supplying water to the Volunteer reservoir and the Queen Anne standpipes. The below-grade structure consists of cast-in-place 12-in. thick reinforced concrete basement walls on continuous footings. The building measures 20-ft. wide by 30-ft. long. The concrete slab roof is supported by three transverse concrete beams and is 13 ft. above the interior slab.

11.36.2 Structure Evaluations

From the original construction documents, the 12-in. basement perimeter walls are reinforced in each face with #5 bars according to the minimum requirements of the current ACI code. All wall reinforcement appears to be doweled adequately to provide acceptable seismic performance. We feel that a Class I-type earthquake will cause some visible cracking in a few members, but judge this structure as having a low vulnerability to seismic damage.

11.36.3 Equipment and Contents Evaluations

Pumps

The two pumps contained within the building are well anchored and have a low vulnerability to damage.

Electrical Control Panels

The motor control panel on the concrete slab in the SW corner of the building has dimensions such that it could topple in a Class I-type earthquake. We recommend that anchorage of the unit to the floor slab be verified.

Telemetry Equipment

The telemetry equipment and two recorders are anchored to wood panels which are in turn anchored to 1 x 4-in. wood sheathing and to the concrete basement walls. This equipment has low vulnerability to seismic damage.

11.36.4 Summary and Conclusions

a) From this preliminary structural investigation, the building is judged to have low vulnerability to a Class I event.

b) The motor control center equipment anchorage should be verified.

c) Table 11.36-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.36-2.

Table 11.36-1

Summary of Preliminary Seismic Evaluations
Volunteer Park Pump Station (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panels	2	1	Verify/supply anchorage

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.36-2

Cost Estimate

Location: Volunteer Park Pump Station

Date Constructed: 1958

Priority: High

Seismic Strengthening Objectives:

1) Prevent control panels from overturning

Upgrade Recommendations:

1) Verify/anchor control panels

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: +30%

11.37 Volunteer Park Gate House and Chlorination Building

11.37.1 Facility Description

The Volunteer Park facility, constructed in 1957, is located at 12th Avenue E. and E. Prospect Street at the south end of the Volunteer Park Reservoir. The structure is approximately shaped like a rectangle measuring 26-ft. wide by 19-ft. long with two half circles on each end with 14-ft. radii. The total length of the structure is about 47 ft. The upper structure is founded on a massive unreinforced concrete base that extends to about 30 ft. below grade at the west end of the building. All of the original window openings have been filled in with block. The original drawings of the structure were not very explicit in detail.

11.37.2 Structure Evaluations

The original construction drawings show 16-in. thick unreinforced concrete walls that extend to 20 ft. above the ground floor to the concrete roof slab above. The concrete roof slab is approximately 6-in. thick and is supported by steel "I" beams placed within the concrete roof section. The connection of the concrete roof slab to the exterior concrete walls appears to be a keyed connection. Thus, the transfer of the roof diaphragm shear to the walls will be accomplished by shear friction. During the inspection, many wall cracks were noticed. This is probably due to the lack of steel reinforcement within the walls. Since the concrete foundation is so massive, we find this portion of the building to have a low vulnerability to seismic damage.

In 1958, an addition to this building was constructed to house the chlorination equipment. This addition is at the eastern portion of the existing building and measures approximately 16 ft. by 23 ft. A new floor slab was constructed at the first floor level that measures 11 ft. by 17 ft. The floor slab appears to be well constructed and has a low vulnerability to seismic damage. An 8-in. block reinforced masonry wall was constructed on the new floor slab to serve as a partition wall between the gate house and the chlorination facilities. This block wall extends to the roof of the existing gate house and has an intermediate reinforced pilaster at the center. Due to its construction, we find this wall to have a low vulnerability to seismic damage.

The presence of the interior block wall improves the stability of this building in an earthquake. Although the concrete walls are unreinforced according to the drawings, it is our opinion that this building has a low vulnerability to seismic damage.

11.37.3 Equipment and Contents Evaluations

Chlorinator and Chlorine Analyzer

This chlorine equipment is located near the center of the chlorine room addition. The chlorinator cabinet is framed out of light weight plastic and is anchored in all four corners. Most of the piping connected to the equipment is plastic, which has some flexibility for movement. The steel piping, however, is not that flexible and could rupture during an earthquake. The reliability of this equipment could be improved by providing flexible couplings on the piping. We find this equipment to have a low vulnerability to seismic damage.

150 lb Chlorine Cylinders

The chlorine cylinders are anchored to the block walls with chains. The cylinders when in use rest on the scale and are also chained to the block walls. These cylinders are considered as having a low vulnerability to damage.

Miscellaneous Equipment

Most other equipment consists of wall mounted electrical panels, telemetry, and chart recorders. This equipment has a low vulnerability to seismic damage.

11.37.4 Summary and Conclusions

a) In this preliminary structural investigation, an inspection was made of the structure and construction drawings were reviewed. The drawings did not provide sufficient detail to allow for a detailed analysis. We recommend that a more in-depth engineering analysis of the structure be performed. With the brief investigation made, it is our opinion that this structure has only a minor vulnerability to seismic damage. Seismic strengthening measures do not appear to be warranted at this time.

b) The equipment within the building has a low vulnerability to seismic damage.

c) Table 11.37-1 summarizes the results of the seismic evaluations. A cost estimate for the recommended additional investigation of the structure is provided in Table 11.37-2.

Table 11.37-1

Summary of Preliminary Seismic Evaluations
Volunteer Park Gate House and Chlorination Building
(High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	More analysis required
2	Chlorinators	1	1	Consider flexible couplings
3	Chlorine cylinders	1	1	O.K.
4	Chlorine building addition	1	1	O.K.
5	Gate House equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.37-2

Cost Estimate

Location: Volunteer Park Gate House and Chlorination Building

Date Constructed: 1957

Priority: High

Additional Investigation Required:

1) Detailed analysis of seismic performance of structure

Cost Estimate:

1)	Engineering	$5,000
	Total:	$5,000

Accuracy of Estimate: ±20%

11.38 Warren Ave. N. Pump Station

11.38.1 Facility Description

The Warren Avenue facility is located on the corner of Warren Ave. N. and Valley St. It supplies water to the 520 zone and the Queen Anne standpipe. The structure, constructed in 1957 is a cast-in-place reinforced concrete building on continuous footings. The building measures 13-ft. wide by 40-ft. long and the north wall of the structure retains earth up to the roof level. The 8-in. thick concrete slab roof is supported by exterior concrete walls and is nearly 12 ft. above the floor slab. The four original window openings each measure 3 ft. 4 in.-wide and are located at the south elevation along with a 3 x 6-ft. door opening. Vent openings in the other three elevations are negligible. No evidence of settling or wall cracking was observed in this building.

11.38.2 Structure Evaluations

From the original construction documents, the 10-in. exterior concrete perimeter walls extend about 4 ft. above the interior floor slab, with 8 in. walls continuing up to the roof slab. The walls are provided with at least #4 bars at 9-in. o.c. and #5 bars at 10-in. o.c., which is adequate reinforcing according to the ATC seismic evaluation sheet. The connection of the walls to the slab diaphragms by doweling appears to be adequate to resist seismic forces. The walls are also doweled into the continuous footings. Although the diaphragm openings in the south elevation wall consist of approximately 50% of its length, an analysis of the shear stress in the walls reveals that only minor cracking should occur in a Class I seismic event.

11.38.3 Equipment and Contents Evaluations

Pumps

Two pumps are contained within the pump station. These pumps rest on concrete pedestals and are anchored with anchor bolts. This equipment, along with the telemetry equipment secured to the concrete wall, has a low vulnerability to earthquake damage.

Electrical Control Panels

The motor control panel that operates the pumps appears to have a low vulnerability to seismic damage if anchorage to the floor slab exists. Anchorage should be verified, and if missing, anchorage using concrete expansion anchors should be provided to secure the panel.

11.38.4 Summary and Conclusions

a) From this preliminary investigation of the original construction documents, it appears that this structure will experience minor structural damage in a Class I earthquake.

b) Anchorage of the motor control panel to the floor slab with anchor bolts should be verified. If missing, the panel should be secured with concrete expansion anchors.

c) Table 11.38-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.38-2.

Table 11.38-1

Summary of Preliminary Seismic Evaluations
Warren Avenue Pump Station (Medium Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps	1	1	Well anchored
3	Electric control panels	2	1	Verify/supply anchorage
4	Telemetry equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.38-2

Cost Estimate

Location: Warren Ave. N. Pump Station

Date Constructed: 1957

Priority: Medium

Seismic Strengthening Objectives:

1) Prevent control panel from overturning

Upgrade Recommendations:

1) Verify/anchor control panel

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 100
2)	Construction Engineering	0
3)	Construction	500
	Subtotal:	600
4)	Sales Tax (8.1%)	40
	Total:	$ 640

Accuracy of Estimate: +30%

11.39 West Seattle Pump Station and Chlorination Building

11.39.1 Facility Description

The West Seattle facility was constructed in 1946 and is located on 8th Ave. S.W. and S.W. Trenton St. It supplies water to the 488 zone as well as to the Myrtle reservoir and the S.W. Charlestown standpipe. The structure is cast-in-place reinforced concrete on continuous footings. The building measures 19-ft. wide by 36-ft. long. The concrete slab roof is supported by three transverse concrete beams and is 10 ft. above the interior slab. The original window openings have been filled in with c.m.u. block. An addition, completed in 1945, provides an "L"-shaped building plan. The addition measures 24 ft. by 36 ft.

11.39.2 Structure Evaluations

From the original construction documents, the 8-in. exterior concrete perimeter walls are generally reinforced with #4 bars at 10-in. o.c., which provide acceptable seismic performance. A Class I type earthquake will impart visible cracking or yielding of a few members but shouldn't affect the structural integrity of the building. Some concrete spalling and cracking could occur between the existing building and the addition since reinforcing details were not provided at the re-entrant corners. Thus, this structure is judged as having a low vulnerability to seismic damage in a Class I-type earthquake.

11.39.3 Equipment and Contents Evaluations

Pumps and Valves

The three pumps contained within the newer building addition are well anchored and have a low vulnerability to damage. Some of the valves and piping rest on cinder block, and we suggest that a permanent concrete pedestal be installed instead.

Electrical Control Panels

Due to their relatively large height and small width, the eastern motor control panels are susceptible to overturning in a seismic event. Anchorage of the units to the floor should be verified. In the absence of adequate floor anchorage, we recommend that the panels be braced to the east concrete wall at the top of the units to prevent overturning during an earthquake.

Chlorine Room Equipment

The chlorine room is located in the S.E. corner of the pump station building and measures 11 ft. by 9 ft. Due to the light plastic frame

construction of the chlorinators, we feel that these units have a low vulnerability to seismic damage.

150 lb Chlorine Cylinders

The chlorine cylinders are anchored to the block walls with chains. The cylinders when in use rest on the scale and are also chained to the block walls. These cylinders are considered as having a low vulnerability to damage.

11.39.4 Summary and Conclusions

a) From this preliminary structural investigation, the structure is judged to have a low vulnerability in a Class I event.

b) The anchorage of the motor control equipment should be verified and, if necessary, anchored to the east wall for stability and prevention of conduit rupture.

c) Table 11.39-1 summarizes the results of the seismic evaluations. An estimate of the remedial upgrade costs is presented in Table 11.39-2.

Table 11.39-1

Summary of Preliminary Seismic Evaluations
West Seattle Pump Station and Chlorination Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Pumps & Valves	2	1	Well anchored; valves require better support
3	Electric control panel	2	1	Verify anchorage/provide brace as necessary
4	Chlorinators	1	1	Consider flexible couplings
5	Chlorine cylinders	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 11.39-2

Cost Estimate

Location: West Seattle Pump Station and Chlorination Building

Date Constructed: 1946

Priority: High

Seismic Strengthening Objectives:

1) Provide permanent concrete pedestals for valves and piping presently resting on cinder block
2) Verify anchorage of east motor control panels
3) Secure chlorine tanks to prevent chlorine tank feeder lines from rupturing

Upgrade Recommendations:

1) Replace cinder block concrete supports with concrete pedestals
2) Verify anchorage of east motor control panels and brace to wall, as necessary
3) Wrap chlorine tanks with strap or construct removable reinforced bollards at the end of the tank rack and side rails paralleling the tank rack on both sides

Assumption: Construction Performed In-House

Cost Estimate:

1)	Engineering	$ 3,000
2)	Construction Engineering	2,000
3)	Construction	5,000
	Subtotal:	10,000
4)	Sales Tax (8.1%)	400
	Total:	$10,400

Accuracy of Estimate: ±35%

11.40 Summary and Conclusions

11.40.1 Geotechnical

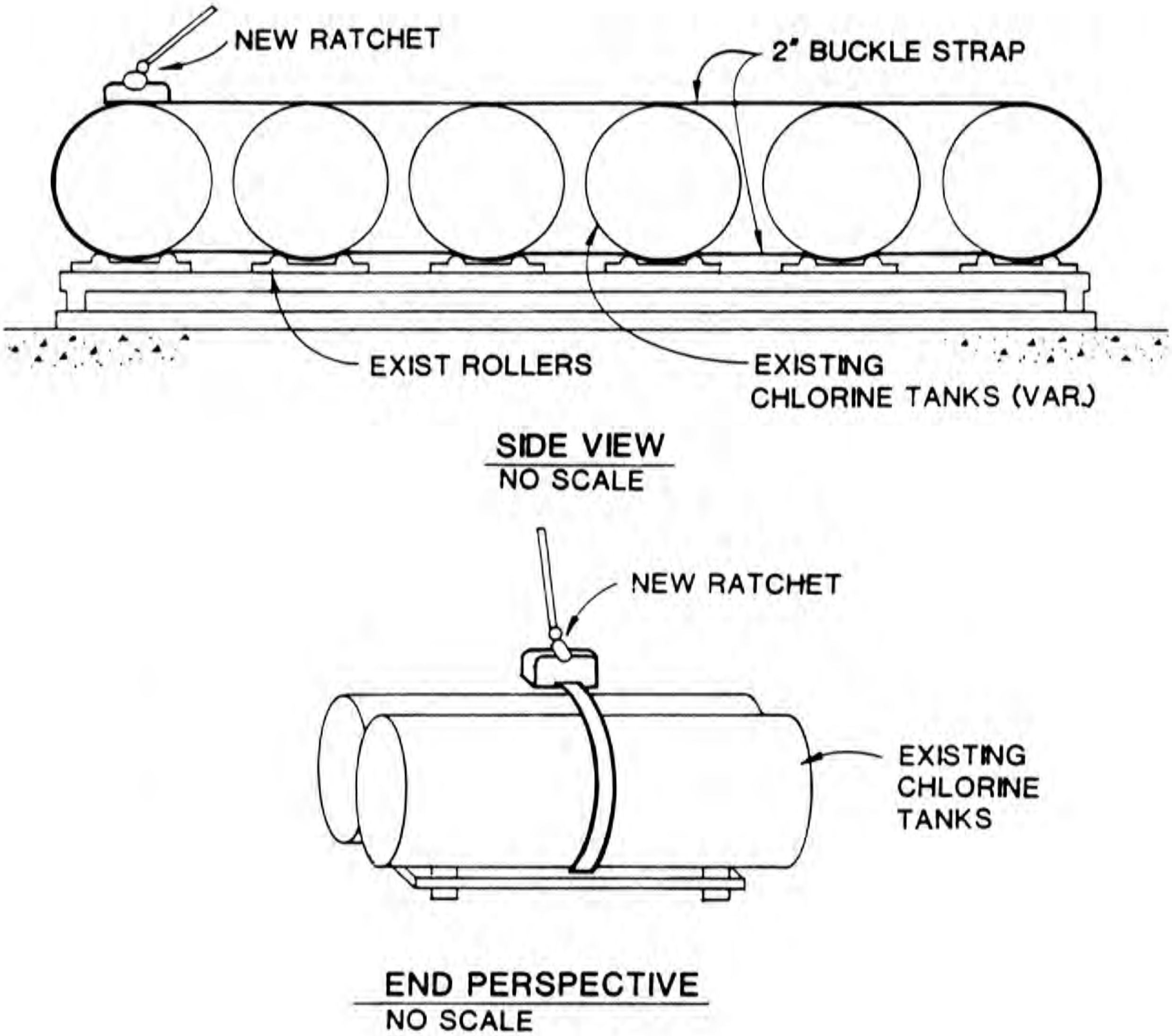
The chlorination facilities and pumping stations are generally located upon competent, glacially consolidated sediments. The competency of these sediments and the topographic relief at sites would generally preclude concerns for major earthquake-induced liquefaction or landslides. Only the S.W. Spokane Street pumping station is located in an area with active soil creep (instability). It is unlikely that any earthquake-induced movement of the hillside in this area would constitute a major threat to the operation of the pump station. Continued creep in the hillside may be detrimental to pipelines leading into the pumping station. As this facility has performed relatively satisfactorily for the past 30 years, any remedial action to improve hillside stability, such as installation of subsurface drainage, would have a relatively low priority for installation.

11.40.2 Structure and Equipment

The evaluation of the pump stations and chlorination facilities was based on a site inspection, a review of the construction drawings, and an evaluation prepared with the assistance of the ATC-14 method for evaluating the seismic resistance of existing buildings. The objective was to identify weak links that might present seismic hazards. In many cases, the construction documents were incomplete or not detailed enough to draw accurate conclusions. In these cases, more investigation and inspection was recommended.

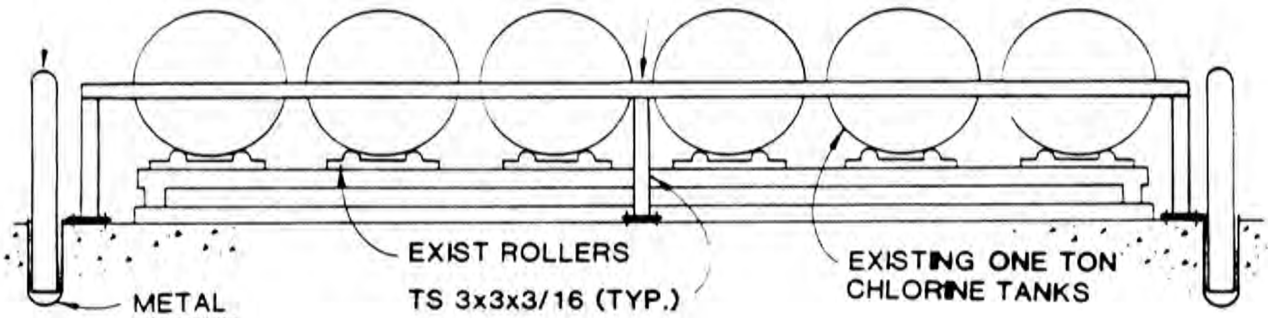
Table 11.40-1 lists those facilities studied, along with their vulnerability to seismic damage, facility priority, estimated repair cost, and an indication as to whether the upgrades can be implemented by the SWD maintenance staff or an outside contractor.

A common potential hazard was found in the chlorination facilities. The large chlorine tanks, supported on racks over scale pits, are not secured. The chlorine tanks could slide off their racks in an earthquake, rupturing the pressurized feeder lines and causing leakage of chlorine gas. Two alternative methods have been proposed for securing the tanks. The first is to wrap a strap around the tanks and tighten with a ratchet. The second is to construct removable reinforced bollards at each end of the tank racks, along with tube steel rail headers which are parallel to the tank rack on both sides. Either of these methods would prevent the tanks from sliding far enough to sever the feeder lines. The costs projected for securing the chlorine tanks in this report assume that both methods are used in combination.

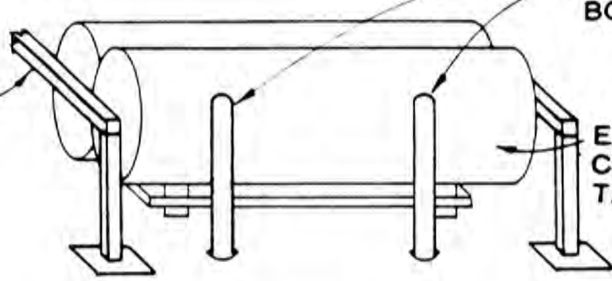


Securing Chlorine Tanks - Strap and Ratchet Arrangement

Figure 11.40-1



SIDE VIEW
NO SCALE



END PERSPECTIVE
NO SCALE

Securing Chlorine Tanks - Removable Bollards and Side Rail Arrangement

Figure 11.40-2

Table 11.40-1

Summary of Upgrade Recommendations for Chlorination Facilities and Pump Stations (Based on 1989 Dollars)

			Estimated Costs								
Facility	Vulnerability[1]	Facility Priority	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
S. Augusta St.	Low	High	$ -0-	$ 100	$ -0-	$ 500	$ 600	$ 40	$ 640	±30%	SWD
Beacon Hill Adm. Bldg.	Moderate	High	5,000	25,600	19,700	49,200	99,500	4,000	103,500	±40%	Contractor
Beacon Hill Sodium Hypo.	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Bitter Lake	High	High	-0-	28,700	16,000	53,000	97,700	4,300	102,000	±35%	Contractor
Bothell Way	Low	High	-0-	200	-0-	700	900	60	960	±30%	SWD
Boulevard Well	Low	High	-0-	100	-0-	500	600	40	640	±30%	SWD
Broadway	Moderate	High	5,000	21,300	9,500	41,700	77,500	3,400	80,900	±35%	Contractor
Burien	Medium	Medium	-0-	20,500	8,700	30,000	59,200	2,400	61,600	±35%	Contractor
Dayton Avenue	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Eastgate	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Fairwood	Medium	Medium	3,000	10,900	6,100	20,000	40,000	1,600	41,600	±35%	Contractor
First Hill	Low	High	-0-	100	-0-	500	600	40	640	±30%	SWD
Foy	Moderate	Medium	-0-	100	-0-	500	600	40	640	±30%	SWD
Green Lake P.S.	Low	Medium	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Green Lake Chlorination	Low	High	4,000	700	100	1,700	6,500	140	6,640	±30%	SWD
Highland Park	Low	Medium	-0-	100	-0-	500	600	40	640	±30%	SWD
Interbay	High	Medium	-0-	21,900	8,900	40,000	70,800	3,200	74,000	±35%	Contractor
Lake Forest	Moderate	High	3,000	700	100	1,700	5,500	140	5,640	±30%	SWD
Lake Hills	Low	High	-0-	100	-0-	500	600	40	640	±30%	SWD
Lincoln Park	Moderate	High	5,000	-0-	-0-	-0-	5,000	-0-	5,000	±20%	
Magnolia Manor	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Maple Leaf Chlorination	Low	High	-0-	700	100	1,700	2,500	140	2,640	±25%	SWD
Maple Leaf P.S.	Moderate	High	3,600	-0-	-0-	-0-	3,600	-0-	3,600	±20%	

Table 11.40-1
(Continued)

			Estimated Costs								
Facility	Vulnerability[1]	Facility Priority	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Maplewood	Moderate	High	$ 3,000	$ 10,500	$ 6,300	$ 15,000	$ 34,800	$ 1,200	$ 36,000	±35%	Contractor
S.W. Myrtle	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
North City	Low	Medium	-0-	200	-0-	1,000	1,200	80	1,280	±30%	SWD
Northgate	Low	Low	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Riverton Hts.	Low	High	-0-	100	-0-	500	600	40	640	±30%	SWD
Roosevelt Way	High	Low	-0-	21,400	10,300	50,000	81,700	4,100	85,800	±35%	Contractor
S.W. Spokane St.	High	Medium	4,000	17,900	6,300	28,000	56,200	2,300	58,500	±35%	Contractor
S.W. Spokane St.(slope)	Low	Low	7,000	-0-	-0-	9,000	16,000	700	16,700	±35%	SWD
Tess Junction	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
S.W. Trenton P.S.	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
S.W. Trenton Chlorination	Low	High	-0-	700	100	1,700	2,500	140	2,640	±25%	SWD
View Ridge	Low	Low	-0-	100	-0-	500	600	40	640	±30%	SWD
Volunteer Park P.S.	Low	High	-0-	100	-0-	500	600	40	640	±30%	SWD
Volunteer Chlorination	Moderate	High	5,000	-0-	-0-	-0-	5,000	-0-	5,000	±20%	
Warren Ave. N.	Low	Medium	-0-	100	-0-	500	600	40	640	±30%	SWD
West Seattle	Low	High	-0-	3,000	2,000	5,000	10,000	400	10,400	±35%	SWD
Totals:			$47,600	$185,900	$94,200	$354,400	$682,100	$28,700	$710,800		

(1) Vulnerability refers to a combined ranking of both the structure and any associated equipment.

12.0 MISCELLANEOUS FACILITIES

12.1 Beacon Reservoir Gate House

12.1.1 Facility Description

The Beacon Gate House facility is located west of the Beacon Reservoirs and was built in 1910. The building measures 26-ft. wide by 28-ft. long, and has two recessed corners at the front elevation. The upper structure is founded on a massive reinforced concrete base that extends to about 30 ft. below grade. The building has heavily reinforced basement walls. The upper structure consists of unreinforced 8 in. brick walls constructed on top of the deep foundation walls. Six penetrations for doors and windows existed in the original design and have since been filled in with block. The height of the upper structure is approximately 12 ft.

12.1.2 Structure Evaluations

The original drawings of the structure were not very explicit in detail. The concrete roof slab varies in thickness from 4 to 6 in. and is supported by 6 in. steel "T" beams. The beams bear on the masonry walls and 4-in. diameter pipe columns. The connection of the concrete roof slab to the exterior concrete walls appears to be a keyed connection. As such, the transfer of the roof diaphragm shear to the walls will be accomplished by shear friction. During the inspection, many wall cracks were noticed, especially at the concrete roof-to-masonry wall interface.

The building has a 3 ft. terra cotta parapet that appears not to be anchored into the wall or roof diaphragm. There is a hazard of the parapet falling off in a major earthquake.

Since the concrete foundation is so massive, we find this foundation portion of the building to have a low vulnerability to seismic damage. It is our opinion that this building overall has a moderate vulnerability to seismic damage. Since it consists of unreinforced brick, we recommend that a more in-depth analysis be completed to assess the seismic risk involved.

12.1.3 Equipment and Contents Evaluations

Miscellaneous Equipment

A couple of wall mounted electrical panels exist. This equipment is considered to have a low vulnerability to seismic damage.

12.1.4 Summary and Conclusions

a) Through this preliminary structural investigation, we have identified a few possible weak links that need to be investigated further. After a more in-depth engineering analysis of some of the items listed in the structure section, some remedial strengthening of the building may be found to be necessary; however, at present, strengthening does not appear to be warranted.

b) The equipment within the building has a low vulnerability to seismic damage.

c) Table 12.1-1 summarizes the results of the seismic evaluations.

Table 12.1-1

Summary of Preliminary Seismic Evaluations
Beacon Reservoir Gate House (High Priority)

Item	Description	Class I	Class II	Comment
1	Gate house	2	1	More analysis required
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

Table 12.1-2

Cost Estimate

Location: Beacon Reservoir Gate House

Date Constructed: 1910

Priority: High

Additional Investigation Required:

1) Detailed analysis of seismic performance of structure

Cost Estimate:

1)	Engineering	$3,000
	Total:	$3,000

Accuracy of Estimate: ±20%

12.2 Beacon Reservoir Telemetry Building

12.2.1 Facility Description

The Beacon Reservoir Telemetry building is located at the east end of the Beacon Hill Reservoirs. The building is a small wood structure measuring 6 ft. by 8 ft. and the date of construction is unknown. The building is painted green and sits on a concrete slab on grade. The roof is built from wood rafters and the walls are 2 x 4 in. studs with wood sheathing. Attached to the walls are several telemetry boxes.

12.2.2 Structure Evaluations

This telemetry building is a light wood-framed building that has a low vulnerability to seismic damage.

12.2.3 Equipment and Contents Evaluations

Telemetry Equipment

The telemetry equipment is well secured to the wood-framed walls and has a low vulnerability to damage.

12.2.4 Summary and Conclusions

a) This building has a low vulnerability to seismic damage.

b) The equipment within the building has a low vulnerability to seismic damage.

c) Table 12.2-1 summarizes the results of the seismic evaluations.

Table 12.2-1

Summary of Preliminary Seismic Evaluations
Beacon Reservoir Telemetry Building (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

12.3 Beacon Valve Chamber (On Inlet)

12.3.1 Facility Description

A 36 in. valve and two 30 in. valves are located in separate valve chambers to the southeast of the Jefferson Park Field House within the median of Beacon Avenue South. Two 16 in. valves are located in two valve chambers within the Beacon Reservoir fenced area, just southeast of the Beacon Reservoir Telemetry building. The five valve chambers constitute the present investigation.

12.3.2 Structure Evaluations

36 Inch Valve

The 36 in. valve is contained in a deep, old brick masonry chamber. The chamber was not entered, so the information gathered is limited. In our opinion, the damage caused by a Class I seismic event would not be major. We are not recommending any seismic upgrading for this valve chamber.

30 Inch Valves

The 30 in. valves are housed in chambers of very similar construction to the 36-in. valve chamber. These brick chambers are approximately 6 ft. in diameter. It is our opinion that these chambers are well built and have a low vulnerability to seismic damage.

16 Inch Valves

The southern-most 16 in. valve is housed in a small, 3 x 3 x 4-ft. deep chamber. This chamber is constructed from concrete blocks and mortar. In our opinion, this chamber has a low vulnerability to seismic damage. The other 16 in. valve is housed in a 9-ft. diameter chamber about 6-ft. deep. The chamber is constructed from old brick masonry walls and is covered with an 8 in. concrete roof. In our opinion, with the information obtained from the inspection, we believe that this valve structure has a low vulnerability to damage.

12.3.3 Summary and Conclusions

a) Each of the Beacon valve chambers has a low vulnerability to seismic damage.

b) Each of the valves, and the associated electrical equipment, has a low vulnerability to seismic damage.

c) Table 12.3-1 summarizes the results of the seismic evaluations.

Table 12.3-1

Summary of Preliminary Seismic Evaluations
Beacon Valve Chamber (High Priority)

Item	Description	Class I	Class II	Comment
1	36" valve & chamber	1	1	O.K.
2	30" valve & chamber	1	1	O.K.
3	16" valve & chamber	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

12.4 Lake Forest Control Building and Valve Chamber

12.4.1 Facility Description

The Lake Forest Park Control Building, located at the south end of the Lake Forest Park Reservoir, is adjacent to a remote controlled 24 in. ball valve. A 54 in. butterfly valve is located in a vault at the south end of the reservoir near the 46th Avenue N.E. intersection. A 12 in. Howell Bunger valve is located under a small wood structure at the southeast corner of the reservoir. Each of these structures was investigated as part of the present study.

12.4.2 Structure Evaluations

Lake Forest Park Control Structure

This structure is constructed from 8-in. thick concrete walls doubly reinforced. The floor is an 8 in. reinforced concrete slab. The roof slab is an 8-in. thick reinforced concrete slab with slab dowels connecting into the walls. The south elevation wall is faced with brick veneer. In our opinion, this structure has a low vulnerability to seismic damage. There is little equipment in the building; the electrical panels and other equipment have a low vulnerability to damage.

24 Inch Ball Valve

The 24 in. ball valve is located in front of the Control Building. The chamber is reinforced concrete and measures 8-ft. wide by 15-ft. long. The chamber is well reinforced and has a low vulnerability to seismic damage.

54 Inch Butterfly Valve

The 54 in. butterfly valve is contained in a reinforced concrete vault that measures approximately 7 ft. by 12 ft. The walls are doubly reinforced and the roof is a 7 in. reinforced concrete slab. The valve is supported on a concrete brick pier. In our opinion, this valve has a low vulnerability to seismic damage.

12 Inch Howell Bunger Valve

The 12 in. Howell Bunger valve, located at the southeast end of the Lake Forest Park Reservoir, is used to stop the flow of water into the reservoir. A reinforced concrete structure is located directly above the valve to provide shelter for the valve equipment. A small wood structure framed from 2 x 4 in. studs covers the valve equipment. In our opinion this concrete and wood structure has a low vulnerability to seismic damage.

12.4.3 Summary and Conclusions

a) Each of these structures was found to have a low vulnerability to seismic damage.

b) Each of the valves, and the accompanying electrical equipment, was found to have a low vulnerability to seismic damage.

c) Table 12.4-1 summarizes the results of the seismic evaluations.

Table 12.4-1

Summary of Preliminary Seismic Evaluations
Lake Forest Control Building (Low Priority) and Valve Chambers (High Priority)

Item	Description	Class I	Class II	Comment
1	Structure	1	1	O.K.
2	Valve Chambers	1	1	O.K.
3	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

12.5 Important Valve Structures and Controls

12.5.1 Beacon & Leo (30 in. BV on CRPL #3)

12.5.1.1 Facility Description

This 30-in. ball valve on Cedar River Pipeline #3 lies in a vault under the asphalt pavement of Beacon Avenue South near South Leo Street. The vault measures 8.8 ft. by 9.25 ft. with a height of about 7 ft. The vault walls and roof are concrete. The flange joint valve is set on a concrete pedestal. There are 2-in. foam collars around the pipe where the pipe enters and exits the wall. The vault contains electrical and telemetry panels.

12.5.1.2 Structure Evaluations

The vault is in good condition, with no visible evidence of cracking or deflection from roof or lateral loads. The schedule of reinforcement and the roof-to-wall connections are unknown; however, it is unlikely that the lateral loads or roof displacement would be sufficient to cause catastrophic failure of the vault in an earthquake.

12.5.1.3 Equipment and Contents Evaluations

The electrical control and telemetry panels and electrical conduit are flush-mounted to the walls with anchor bolts. The anchorages are sufficient.

12.5.1.4 Summary and Conclusions

This structure is in good condition; in our opinion, only minor structural damage such as cracking is likely to result from an earthquake. No remedial measures are recommended.

12.5.2 Beacon & Jefferson Field House (24 in. BV on CRPL #1 to #2)

12.5.2.1 Facility Description

This 24-in. butterfly valve on the Cedar River Pipeline is located in the median of Beacon Avenue South opposite the north side of the Jefferson Field House. The valve lies in a 12 ft. by 9 ft. concrete vault with a beam-and-slab concrete roof. The flanged valve is supported by a concrete pedestal.

12.5.2.2 Structure Evaluations

There are two roof beams spaced at 6 ft. 9 in. o.c. along the long axis of the vault. In addition, there are two roof beams along the transverse axis of the vault, spaced at 5 ft. 6 in. o.c. The condition of the vault is not good; exposed rusted reinforcing bar is in need of repair. However, the structure is expected to incur only minor damage in an earthquake, such as cracking.

12.5.2.3 Equipment and Contents Evaluation

The valve and piping is not expected to be significantly damaged in an earthquake. However, the valve is rusting, and routine maintenance such as painting of the valve and flange bolts should be performed.

12.5.2.4 Summary and Conclusions

No seismic strengthening measures are required at this structure; however, routine maintenance should be performed to prevent further rusting of reinforcing bar and the valve.

12.5.3 Bothell Way Isolation Valve

12.5.3.1 Facility Description

The Bothell Way isolation valve is located adjacent to the Bothell Way pump station at N.E. 82nd Street and Lake City Way (formerly Bothell Way). The 54 in. isolation valve chamber lies under the street of N.E. 82nd Street, but is accessed from within the Bothell Way pump station in an alcove of the main room. The structure was designed in 1955.

The isolation valve chamber is about 19-ft. long and the pipe spans the room width of 7 ft. The chamber has concrete walls. The valve is basically unsupported; however, support is not necessary for the short span.

The roof of the alcove is the concrete street slab. The roof is subject to live loads from the street traffic.

12.5.3.2 Structure Evaluations

According to the plans, the roof of the isolation valve chamber is designed for 12,000 psf axle loads. The roof, walls and floor are each 8-in. reinforced concrete. The chamber has a low vulnerability to seismic damage.

12.5.3.3 Equipment and Contents Evaluations

The valve and controls are adequately secured, and have a low vulnerability to seismic damage.

12.5.3.4 Summary and Conclusions

The valve chamber, valve and controls all have a low vulnerability to seismic damage. No seismic strengthening measures are recommended.

12.5.4 Cedar River Wye Vaults (36 in. GV and 60 in. BFV)

12.5.4.1 Facility Description

Two valves are located at a wye junction of Cedar River Pipeline No. 4 about 400 yards east of 126th Avenue S.E. and S.E. 160th Street in the Fairwood area. One is a 60 in. butterfly valve on the westerly running branch; the other is a 36 in. butterfly valve on the northwesterly running branch. The valves are housed in separate, though similar, underground vaults.

Both vaults have walls built of 4 x 8 x 16-in. solid concrete mortar units, and 8-1/2 in. roof slabs that are keyed about an inch deep to fit into the top of the walls. Both have dirt floors. Electrical control boxes and conduits are flush-mounted on the walls in each vault. Both valves are set on 16 x 16-in. grouted c.m.u. plinths.

The vault containing the 60 in. butterfly valve is about 13 ft. long by 5 ft. 9 in. wide, with the pipe spanning the shorter distance. The west wall where the pipe enters the wall is concrete in the form of a collar about a foot and a half wide. The rest of the wall is c.m.u. The height of the vault is 10-1/2 ft. The vault containing the 36 in. butterfly valve is about 7 ft. 9 in. by 10 ft. 4 in., with a height of 9 ft. 7 in. The pipe spans the shorter width.

12.5.4.2 Structure Evaluations

The vaults are in good condition. While there does not to appear to be reinforcing bar in the walls, the mortar in the walls is generally good. Wall stability is increased by the method used to key the roof slabs. In our opinion, no seismic strengthening of these vaults is required.

12.5.4.3 Equipment and Contents Evaluations

The equipment in the vaults is limited to the valves and air valves, and the electrical control equipment and conduit. The control equipment is adequately mounted. Maintenance painting of the 60 in. butterfly valve and 4 in. air valve is recommended to prevent continued rusting.

12.5.4.4 Summary and Conclusions

The vaults housing the 60 in. butterfly valve and the 36 in. butterfly valve are in good condition, and have low vulnerability to seismic damage. Similarly, the valves and controls do not require seismic strengthening. Maintenance painting of the 60 in. butterfly valve and 4 in. air valve is recommended to prevent continued rusting.

12.5.5 Mercer Island Pipeline Vault (24 in. BV)

12.5.5.1 Facility Description

The 24-in. gate valve is located in an unfinished vault in the Newport area near 120th Avenue S.E. and S.E. 35th Street. The vault is located in open ground set back about 50 ft. from the top of a steep vegetated slope. This valve is presently being replaced with a 24-in. remote controlled ball valve.

The vault is L-shaped, with the 24-in. pipeline running through the long axis and the gate value sitting sideways in the shorter transverse axis. The long axis is about 17 ft. by 6 ft.; the transverse axis extends 3-1/2 ft. to one side with a width of 5-1/2 ft. Vault height is about 6 ft., with a dirt floor.

The walls of the vault are constructed of 8 x 16 x 4-in. solid concrete mortar units. The upper foot or so of the walls have not yet been grouted. The present vault configuration is a modification of an older concrete mortar unit vault designed to accommodate the 24-in. gate valve. Part of the roof has been temporarily covered with an 8-in. precast concrete slab, which was installed to cover the open hole until construction could be completed, and is not grouted into place.

12.5.5.2 Structure Evaluations

The plans for the vault do not indicate any reinforcement of the concrete mortar units which form the chamber walls. This is not of particular concern from a seismic standpoint, provided the walls are properly mortared. However, the mortar work of the walls in the vault is of poor quality, with incomplete and uneven buttering, large gaps and questionable mix. On the older portion of the walls, it appears that the walls may have started to buckle at some time. Subsequent to construction, four tie-backs were constructed, two in each longitudinal wall. The walls can sustain relatively little shear stress.

When the structure is completed, reconstructive mortaring of the existing walls should be performed to fill in the gaps. Some reconstruction of the worst sections of existing masonry should be included. Doweling of the roof slabs to the walls can help stabilize the walls. The roof slab design, as shown on the plans, is well reinforced.

12.5.5.3 Equipment and Contents Evaluations

There is no equipment in the vault other than the gate valve and a 2-in. blowoff. The main concern is damage to the valve and blowoff if there is structural failure. The valve rests on a 16-in. square mortared concrete block pedestal. The valve is flange-jointed.

The pipe, valve and blowoff would not be expected to sustain significant damage in an earthquake unless the slab roof were to become separated from the walls and collapse. If the vault is completed in a manner that inhibits large displacements of the roof slab, then damage would not be expected to the piping.

12.5.5.4 Summary and Conclusions

The existing walls of this unfinished vault can sustain relatively little shear stress. Completion of the structure should include remedial grouting and replacement of the worst sections of the existing walls. Doweling of the roof slabs to the walls would strengthen the shear resistance of the walls.

12.5.6 Summary and Conclusions

Table 12.5-1 provides a summary of the important valve structures and controls which were evaluated and reported upon in Section 12.5, along with their vulnerability to seismic damage for both the Class I and Class II earthquakes. All of the facilities were found to be of low vulnerability, with only minor recommendations provided for remedial maintenance repairs of selected vaults and valves.

Table 12.5-1

Summary of Preliminary Seismic Evaluations
Important Valve Structures and Controls

Item	Description	Class I	Class II	Comment
1	Beacon & Leo			
	Vault	1	1	O.K.
	Valve	1	1	O.K.
	Equipment	1	1	O.K.
2	Beacon & Jefferson			
	Vault	1	1	Maintenance required
	Valve	1	1	Maintenance required
	Equipment	1	1	O.K.
3	Bothell Way			
	Vault	1	1	O.K.
	Valve	1	1	O.K.
	Equipment	1	1	O.K.
4	Cedar River Wye			
	Vaults	1	1	O.K.
	Valves	1	1	Maintenance required
	Equipment	1	1	O.K.
5	Mercer Island Pipeline			
	Vault	1	1	Remedial grouting required
	Valve	1	1	O.K.
	Equipment	1	1	O.K.

Evaluation Categories:

1 = Low Vulnerability (operable)
2 = Moderate Vulnerability (operable with some repairs)
3 = High Vulnerability (non-operable)

12.6 Summary and Conclusions

Table 12.6-1 provides a summary of those miscellaneous facilities studied, along with their vulnerability to seismic damage, facility priority, and estimated repair cost.

Table 12.6-1

Summary of Upgrade Recommendations
Miscellaneous Facilities
(Based on 1989 Dollars)

			Estimated Costs								
Facility	Vulnerability[1]	Facility Priority	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate	Construction
Beacon Reservoir Gate House	Moderate	High	$ 3,000	$ -0-	$ -0-	$ -0-	$ 3,000	$ -0-	$ 3,000	±20%	
Beacon Reservoir Telemetry Building	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Beacon Valve Chamber	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Lake Forest Control Building	Low	Low	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
and Valve Chamber		High									
Beacon & Leo Vault and Valves	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Beacon & Jefferson Vault and Valves	Low	High	-0-	200	100	2,000	2,300	150	2,450	±25%	SWD
Bothell Way Isolation Valve	Low	High	-0-	-0-	-0-	-0-	-0-	-0-	-0-		
Cedar River Wye Vaults and Valves	Low	High	-0-	200	100	2,000	2,300	150	2,450	±25	SWD
Mercer Island Pipeline Vault and Valve	Low	High	-0-	600	200	3,000	3,800	300	4,100	±25%	SWD
Totals:			$3,000	$1,000	$400	$7,000	$11,400	$600	$12,000		

(1) Vulnerability refers to a combined ranking of both the structure and any associated equipment.

13.0 MAJOR SUPPLY AND DISTRIBUTION PIPELINES

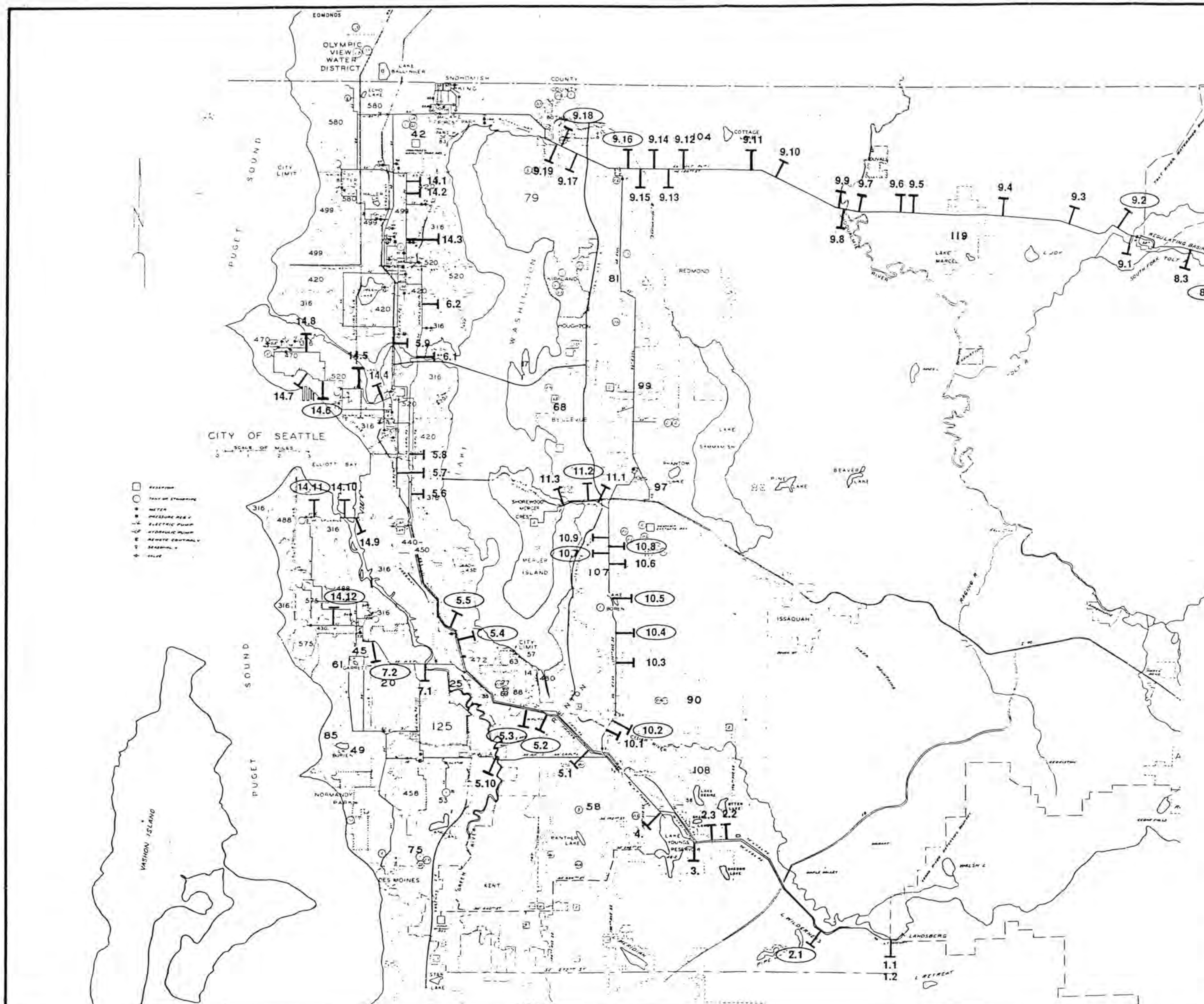
The factors that are detrimental to the earthquake performance of an underground lifeline system include the following, as listed in descending order of importance: faulting, landsliding, and lateral spreading. These phenomena are all of primary importance to underground structures in that these processes may result in large lateral ground deformations which could potentially rupture pipelines. Processes of secondary importance to the earthquake performance of underground lifelines include, in descending order of importance: liquefaction, differential compaction, and ground shaking effects. These processes are of secondary importance as they typically involve a lesser amount of ground strain and, hence, are typically less damaging to underground utilities.

While excessive, earthquake-induced ground strains in stable soil deposits have resulted in some pipeline damage during historic earthquakes, this amount of damage is typically insignificant when compared to damage resulting from faulting, landsliding, or lateral spreading. It has been our experience with major lifeline utilities that the compressional and bending strains induced by surface waves in stable soil deposits seldom control the design of pipelines or utilities in the Pacific Northwest. Therefore, ground instability was considered as the primary factor in evaluating the seismic performance of the pipelines. Under this scenario, zones of significant ground instability would most likely result in pipeline rupture.

Considering the above mechanisms, a geological site reconnaissance was conducted at select locations along the major supply and distribution pipelines to identify zones that may undergo substantial ground movement during a Class I earthquake. These potentially high hazardous zones were identified from the seismic hazard map (Landslide Hazard - Figure 4.3-1) and from the liquefaction hazard map (Figure 4.3-2). As mentioned in Section 4.3, it is unlikely that major pipelines in the Seattle water distribution system would be affected by ground rupture as a result of earthquake faulting.

Conclusions derived on the performance of the various pipelines were based strictly upon our visual site reconnaissance, our review of facility drawings and subsurface information data supplied by the Seattle Water Department, and our knowledge of the subsurface soils in the various service areas. Structural evaluations of the elevated portions of the water lines were not performed. However, instances where pipelines may have inadequate support were noted during the individual field reconnaissances.

The findings of the field reconnaissances are discussed in the following sections of this chapter. Individual observations at discrete points along the pipelines are referenced by paragraph number on Figure 13-1.



LEGEND

10.2 — Pipeline Segments Reviewed in Field and Discussed in Chapter 13 of Report

Specific Comment (Circled where action recommended)

Pipeline Subsection in Chapter 13 of Report

Seattle Water Department
Seattle, Washington

PIPELINE RECONNAISSANCE MAP

May 1989 W-5023-30

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. 13-1

14.0 ELEVATED TANKS

14.1 Facility Overview

The Seattle Water Department has seven elevated water tanks ranging in capacity from 500,000 to 2,000,000 gallons (Table 14.1-1). The tanks are located throughout the urban area on five high ground locations. Because of their size and location, these tanks must maintain their integrity during and following a major earthquake. Their failure could result in catastrophic flooding and potential loss of life.

The tanks are a fundamental component of the water system lifeline. As such, they must remain operable after a major earthquake to provide fire fighting capabilities, maintain sanitary conditions and to provide drinking water. Therefore all elevated tanks are considered to be high priority facilities for purposes of this study.

Out of the seven tanks, one was constructed in 1919 and the other six were constructed between 1946 and 1959. They are all made of steel. Lateral load capacity is provided by diagonal steel braces.

14.2 Geologic Hazard Assessment

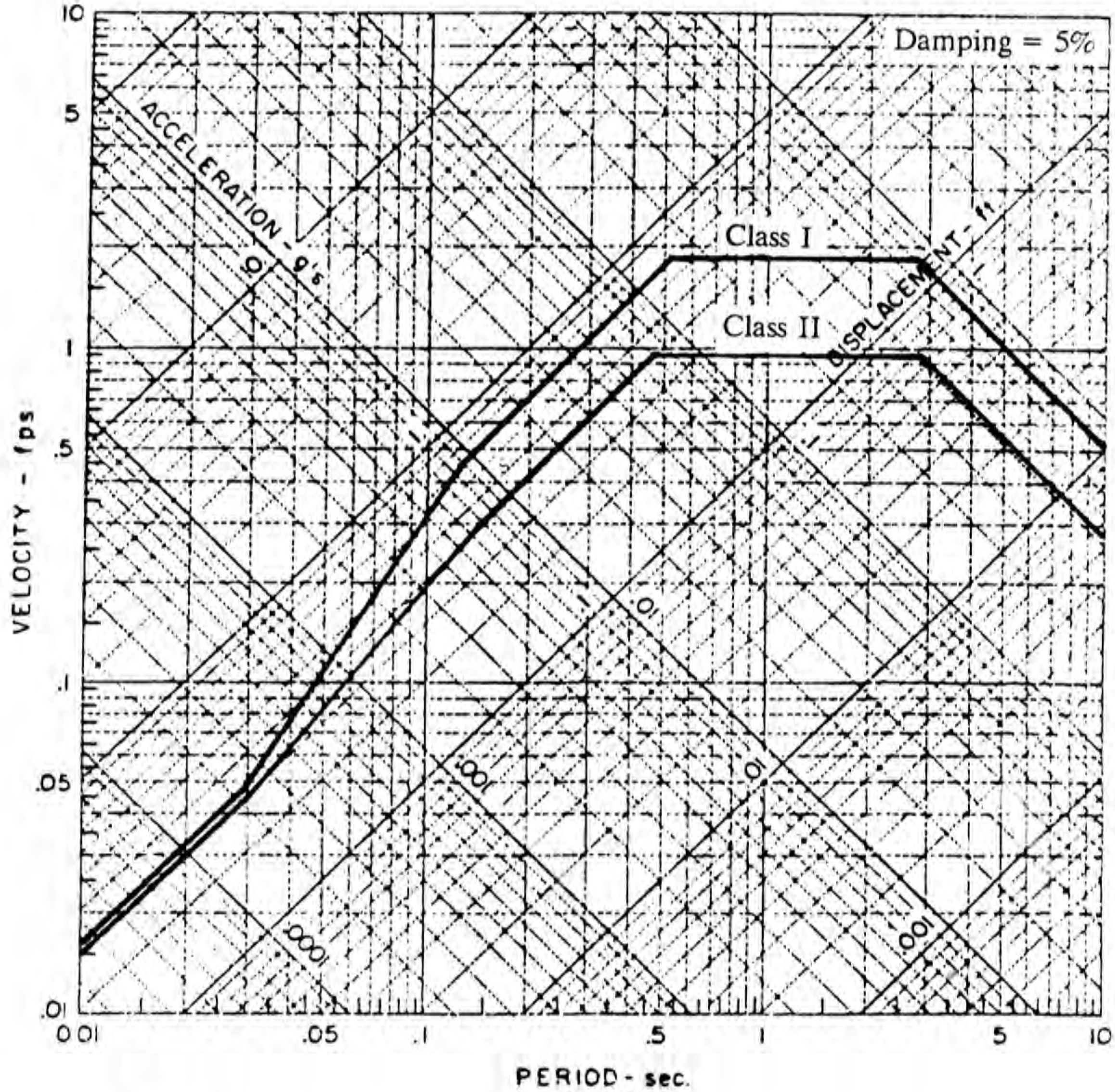
All of the elevated tanks are located in seismic zone II as indicated on the seismic hazard map of Figure 4.3-1. [Figure 14.2-1 shows the seismic design spectra for the Class I and Class II earthquakes.] The subsurface soils at most tank locations consist of competent, glacially consolidated sediments which are not susceptible to liquefaction or earthquake-induced landsliding.

While most of the tanks would appear to be founded upon native, glacially consolidated sediments, it would appear that the Magnolia Bluff and S.W. Myrtle tanks may possibly be founded upon shallow fill soils. Although borings were not available to confirm the composition and thickness of the soils adjacent to these tanks, the absence of any major signs of distress to the foundations of the tanks or adjacent structures would lead us to conclude that any potential fill in these areas is relatively competent. Also, because of the lack of any subsurface soils information at these sites, it was not possible to estimate potential earthquake-induced settlements beneath the foundations. At the time of the rehabilitation of these two tanks, geotechnical investigations should be performed to confirm the adequacy and extent of the fill under these tanks.

Table 14.1-1

Seattle Water Department
Elevated Water Storage Tanks

Name	Location	Date of Construction	Overflow Elevation	Capacity in Gallons at Overflow
Beverly Park	4th Ave. S.W. & S.W. 111th Street	1959	575	2,000,000
Magnolia Bluff	38th Ave. W. & W. Prosper Street	1947	470	1,000,000
Maple Leaf	Roosevelt Way N.E. & N.E. 86th Street	1949	520	1,000,000
S.W. Myrtle #1	36th Ave. S.W. & S.W. Myrtle Street	1919	575	500,000
S.W. Myrtle #2	35th Ave. S.W. & S.W. Myrtle Street	1946	575	1,000,000
Richmond Highlands #1	Fremont Ave. N. & N. 195th Street	1954	580	1,000,000
Richmond Highlands #2	Fremont Ave. N. & N. 195th Street	1958	580	2,000,000
		Total Capacity :		8,500,000



Notes:

1. Spectra are for free-field, horizontal motions at ground surface.
2. Vertical spectra correspond to two-thirds of the above values.
3. Spectra are for Class I earthquake and Zone III conditions.
4. Use 75% and 85% of the above values for Zones I and II, respectively.
5. Spectra for Class II earthquake equal 62% of the above values.

Seismic Response Spectra for Class I and Class II Earthquakes

Figure 14.2-1

14.3 Beverly Park

14.3.1 Facility Description

The Beverly Park elevated water storage tank, built in 1959, has a capacity of 2,000,000 gallons. The structure is shown in Figures 14.3-1 and 14.3-2. The elevated tank shell is supported at its equator by fourteen (14) braced columns with a diameter of 48 in. and a cross sectional area of approximately 70 sq. in. The braces have a cross section of 22 sq. in. There is a set of seven (7) unbraced interior columns and a central riser with a diameter of 72 in. and a cross section of approximately 56 sq. in. The columns are supported by spread footings with tie beams along the outside perimeter only.

The structure has a diameter of approximately 103 ft., and the center of gravity for the structure and contents is located at 96 ft. above grade level.

14.3.2 Analysis Procedure

The computer program ANSYS was used for analysis of the tank. Using this program, a three dimensional model of the tank was developed as shown in Figure 14.3-3. The tank shell connection at the top of the columns was modeled by coupling the horizontal displacements at the top of the columns, and using an equivalent ring beam to restrain the rotations. This assumption is justified because at those connections forces are transferred primarily along the tangential surface of the tank. Along this tangential direction the tank wall is axially very rigid. It should also be noted that elevated tanks are "long period" structures and thus, any modeling assumptions which overestimate their rigidity are conservative because they result in higher spectral accelerations.

Only half of the area of the braces was included in the model. This accounts for the braces not being effective in compression. Column stresses were based on the axial reaction at the base of the footings. In this manner, the column stresses were not artificially reduced by the braces taking some axial load.

There are only two anchor bolts at the base of each column. These connections are modeled as hinges. The riser has four anchor bolts and a small spread footing. Rotational springs were modeled at the base of the riser.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring. The weight of the water is distributed to the columns and riser in accordance to their tributary area.

14.3.3 Analysis Results

The fundamental period of vibration of the tank is 1.22 seconds, and the fluid sloshing period is 5.4 seconds. This places the tank period in the descending branch of the response spectra (Figure 14.2-1). Any softening of the tank (i.e. yielding) will contribute to lowering the effective seismic loads in the structure.

The lateral displacement computed in the elastic analysis is 3.5 inches. Stress ratios for critical elements are presented in Table 14.3-1.

In our opinion, the structure will remain operable after the Class I earthquake. Yielding of the braces and rotation of the riser footing will contribute to softening of the structure and will reduce the earthquake load. Local failure of the riser is not likely to compromise the overall behavior of the structure because of the large number of columns providing redundancy to the system.

Table 14.3-2 shows a comparison of design base shears using the 1988 UBC Code and the Class I and the Class II earthquakes.

14.3.4 Upgrade Recommendations

In order to remain operable when subjected to a Class I earthquake, the supporting structure of the Beverly Park tank will have to rely on ductility and redundancy. In doing so, the structure will undergo a fair amount of racking and damage.

Accordingly, in order to mitigate the seismic vulnerability of this structure, it is necessary to improve its lateral load capacity. A cost effective strategy to achieve this is to introduce diagonal braces in the interior columns (i.e. introduce new, 4-in. diameter diagonal braces in a similar arrangement to the outer braces). Braces in the exterior columns cannot be as readily strengthened because the existing footings are marginally overstressed.

The estimated cost of bracing the interior columns of the Beverly Park elevated water tank, as shown in Table 14.10-2, is $305,600.

Table 14.3-1

Beverly Park Elevated Water Storage Tank (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	1.04	Minor yielding
Riser (tension in bending)	1.24	Yielding
Riser (compression)	1.48	Local buckling
Interior Columns (tension)	1.17	Minor yielding
Exterior Columns (tension)	0.87	Adequate
Tank Shell	0.46	Adequate
Brace Connections	1.00	Adequate
Column to Tank Shell Connections	0.53	Adequate
Column Base Connections	<1.00	Adequate
Soil Under Columns	1.10 (allowable)	Minor overstress, limited by braces yielding
Soil Under Riser	2.40 (allowable)	Rotation of riser footing expected

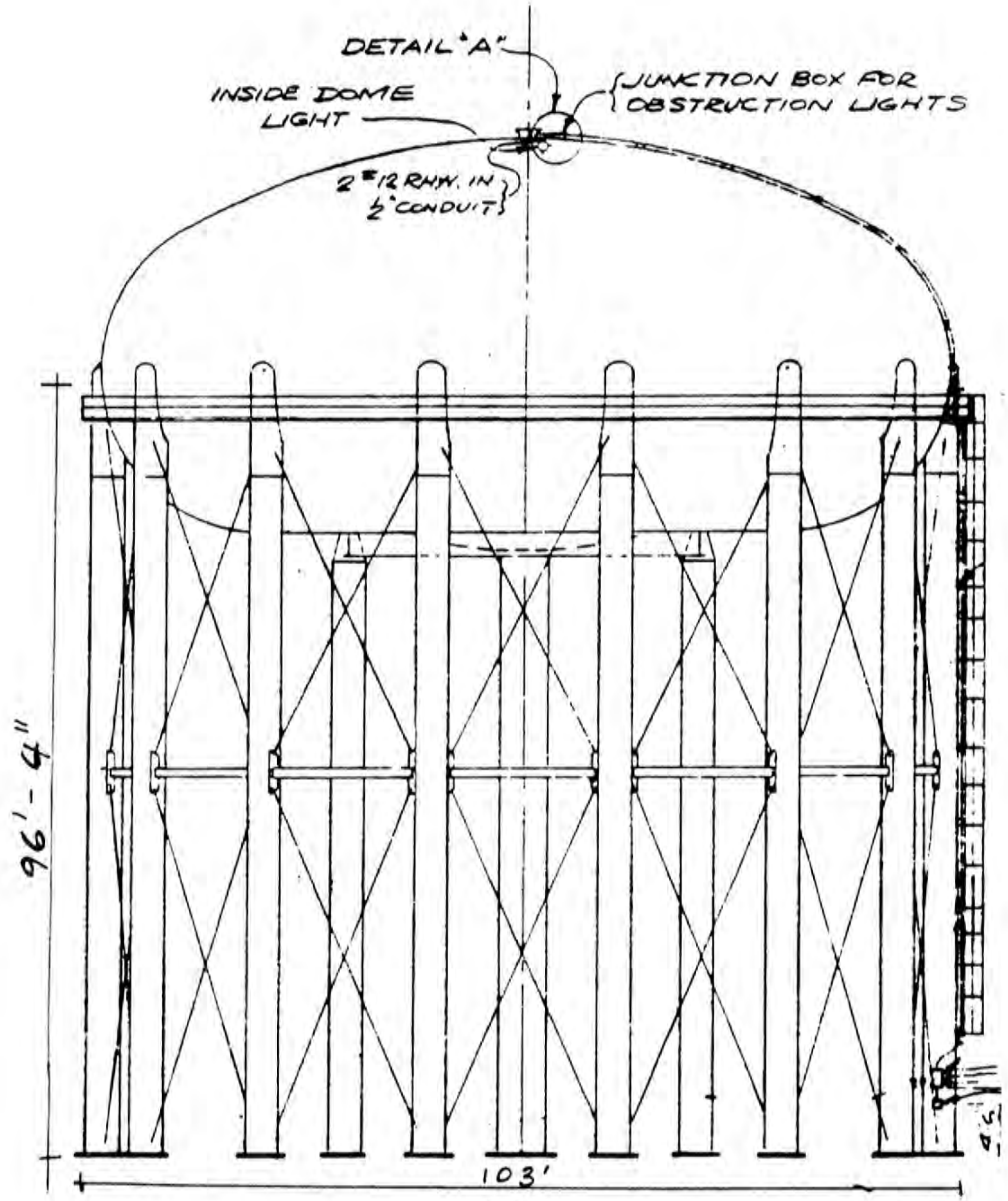
Elevated Tank Status: **Operable**

Estimated Probability of Failure: **10%**

Table 14.3-2

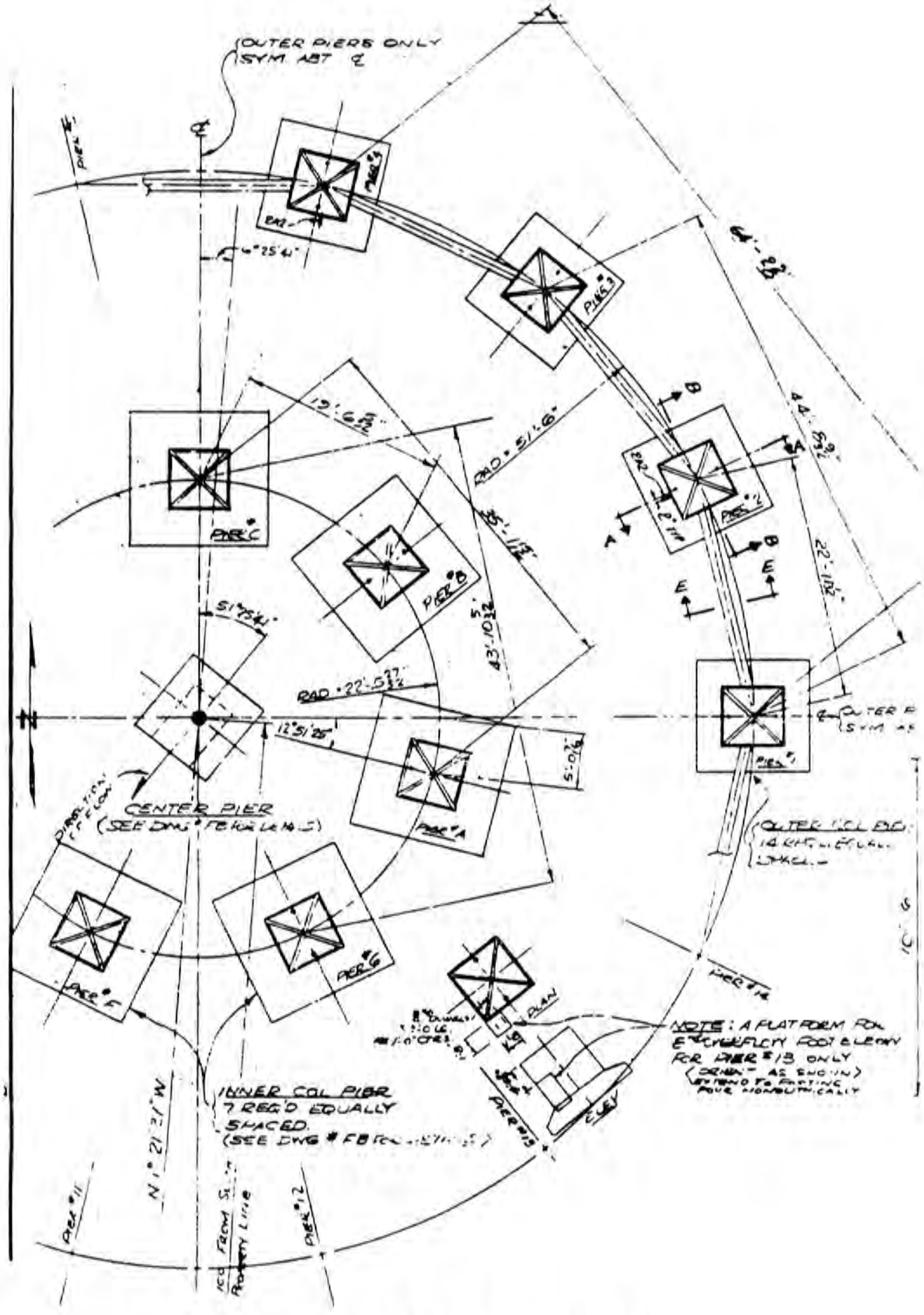
Beverly Park Elevated Water Storage Tank (High Priority)
Design Base Shear Comparison

1988 UBC	3,392 kips
Class I Earthquake	3,083 kips
Class II Earthquake	1,911 kips



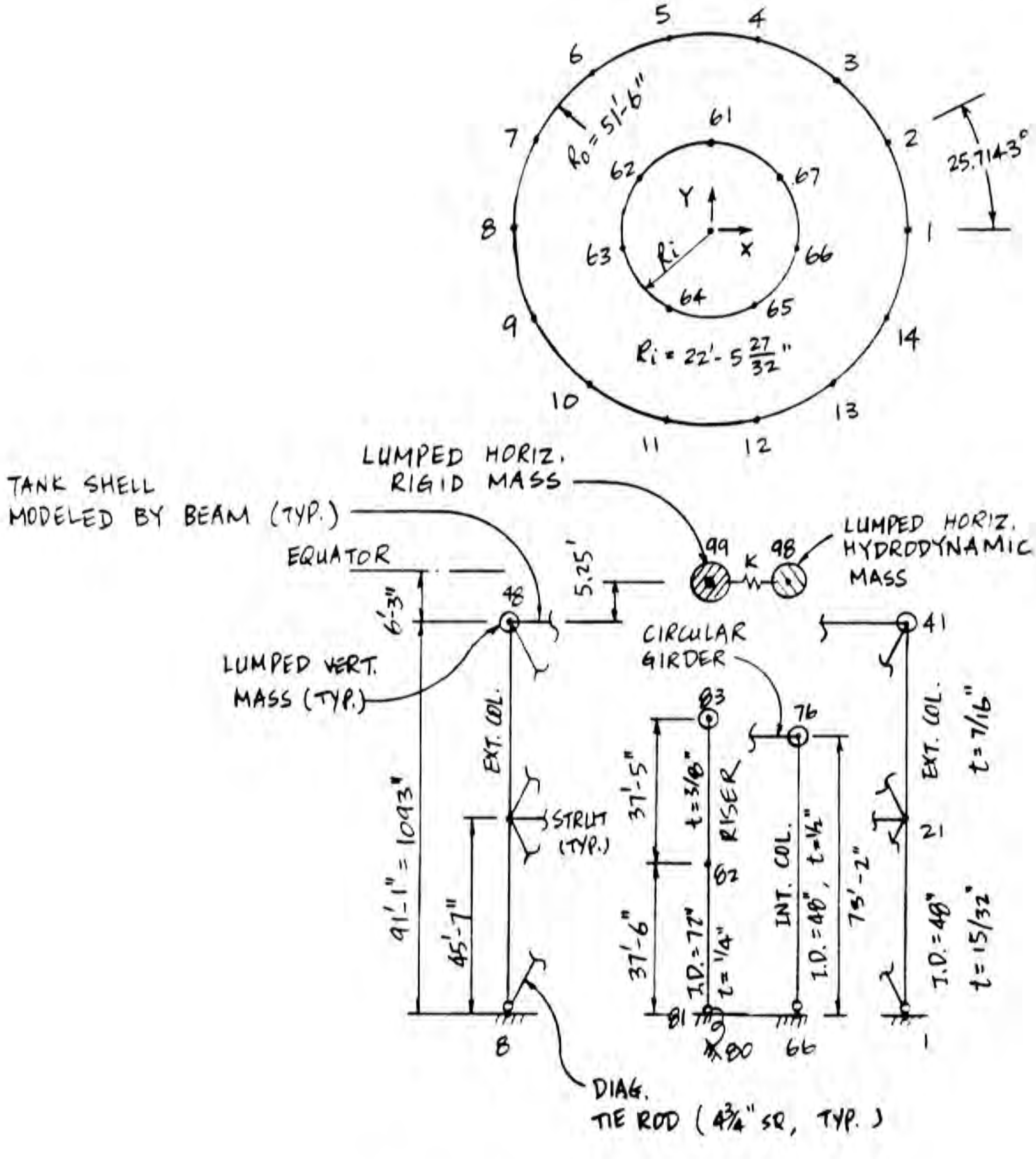
Beverly Park Elevated Water Storage Tank
Elevation View

Figure 14.3-1



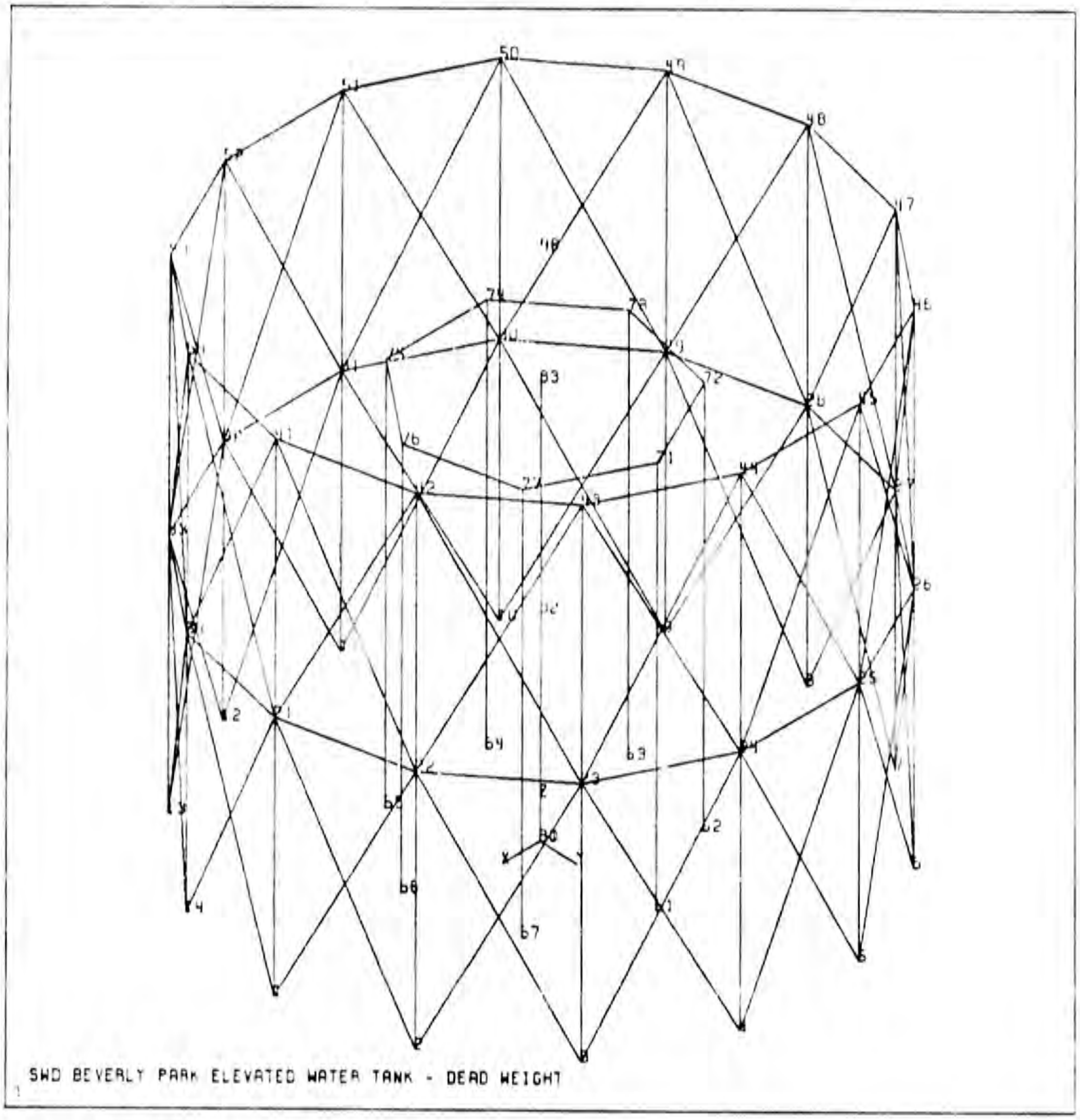
Beverly Park Elevated Water Storage Tank
Plan View

Figure 14.3-2



Beverly Park Elevated Water Storage Tank
Mathematical Model

Figure 14.3-3



Beverly Park Elevated Water Storage Tank
Computer Plot

Figure 14.3-4

14.4 Magnolia Bluff

14.4.1 Facility Description

The Magnolia Bluff elevated water storage tank, built in 1947, has a capacity of 1,000,000 gallons. The structure is shown in Figures 14.4-1 to 14.4-4. The elevated tank shell is supported by radial girders resting on twelve (12) braced columns with a diameter of 48 in. and a cross sectional area of approximately 48 sq. in. The braces have a cross section of 2 sq. in. There is a central riser with a diameter of 96 in. and a cross section of approximately 139 sq. in. The central riser anchorage conditions were not specified in the available drawings. The columns are supported by spread footings with tie beams along the outside perimeter.

The structure has a diameter of approximately 66 ft., and the center of gravity for the structure and contents is located at 88 ft. above grade level.

14.4.2 Analysis

The computer program ANSYS was used in the analysis. The three dimensional model is shown in Figure 14.4-3. The tank shell connection at the top of the columns was modeled by coupling the horizontal radial beams to the top of the columns, and using constraint equations to couple the horizontal displacements without further restraining the rotations. This assumption is justified because at those connections forces are transferred primarily by shear along the tangential surface of the tank. Along this tangential direction the tank shell is very rigid.

Only half of the brace area is included in the model. This approach accounts for the braces not being effective in compression. Column stresses were computed on the basis of axial reactions at the base of the footings. In this manner the column stresses are not artificially reduced by the brace elements taking some axial load.

The connections at the base of each column are modeled as hinges. The riser has no anchor bolts. Rotational springs were modeled at the base of the riser. The water content was modeled using Housner's method. The weight of the water was distributed to the columns and riser in accordance to their tributary area.

14.4.3 Analysis Results

The fundamental period of vibration of the tank is 1.8 seconds, and the fluid sloshing period is 5.2 seconds. This places the tank period in the descending branch of the response spectra (Figure 14.2-1). Any softening of the tank (i.e. yielding) will contribute to lowering the

effective seismic loads in the structure. The expected lateral displacement from the elastic analysis is 4.8 in.

Stress ratios for critical elements are presented in Tables 14.4-1 and 14.4-2. In our opinion, the structure will not remain operable after the Class I earthquake. Yielding of the braces and columns will contribute to instability of the structure.

The structure will remain operable after the Class II earthquake. Moderate yielding of the braces will take place resulting in lower effective loads in the structure.

A comparison of design base shears computed using the 1988 UBC Code and the Class I and Class II earthquakes is provided in Table 4.4-3.

14.4.4 Upgrade Recommendations

This structure requires strengthening of the diagonal braces. It is recommended that the cross-sectional area of the braces be increased by 2 sq. in. to increase their lateral load capacity. In order for the brace strengthening to be effective, the columns should also be strengthened by increasing their cross-sectional area by 28 sq. in.

The estimated cost of upgrading the Magnolia Bluff elevated water storage tank, as shown in Table 14.10-2, is $500,200.

Table 14.4-1

Magnolia Bluff Elevated Water Storage Tank (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	1.80	Yielding
Riser (tension in bending)	1.05	Minor yielding
Riser (compression)	1.08	Local buckling
Columns	1.30	Yielding and buckling
Brace Connections	0.11	Adequate, limited by braces yielding
Column Base Connections	<1.00	Adequate
Radial Girder	0.98	Adequate
Soil Under Columns	0.80 (allowable)	Limited by braces yielding
Soil Under Riser	0.55 (allowable)	Limited by anchor

Elevated Tank Status: **Not Operable**

Estimated Probability of Failure: **70%**

Table 14.4-2

Magnolia Bluff Elevated Water Storage Tank (High Priority) Analysis Results

Class II Earthquake

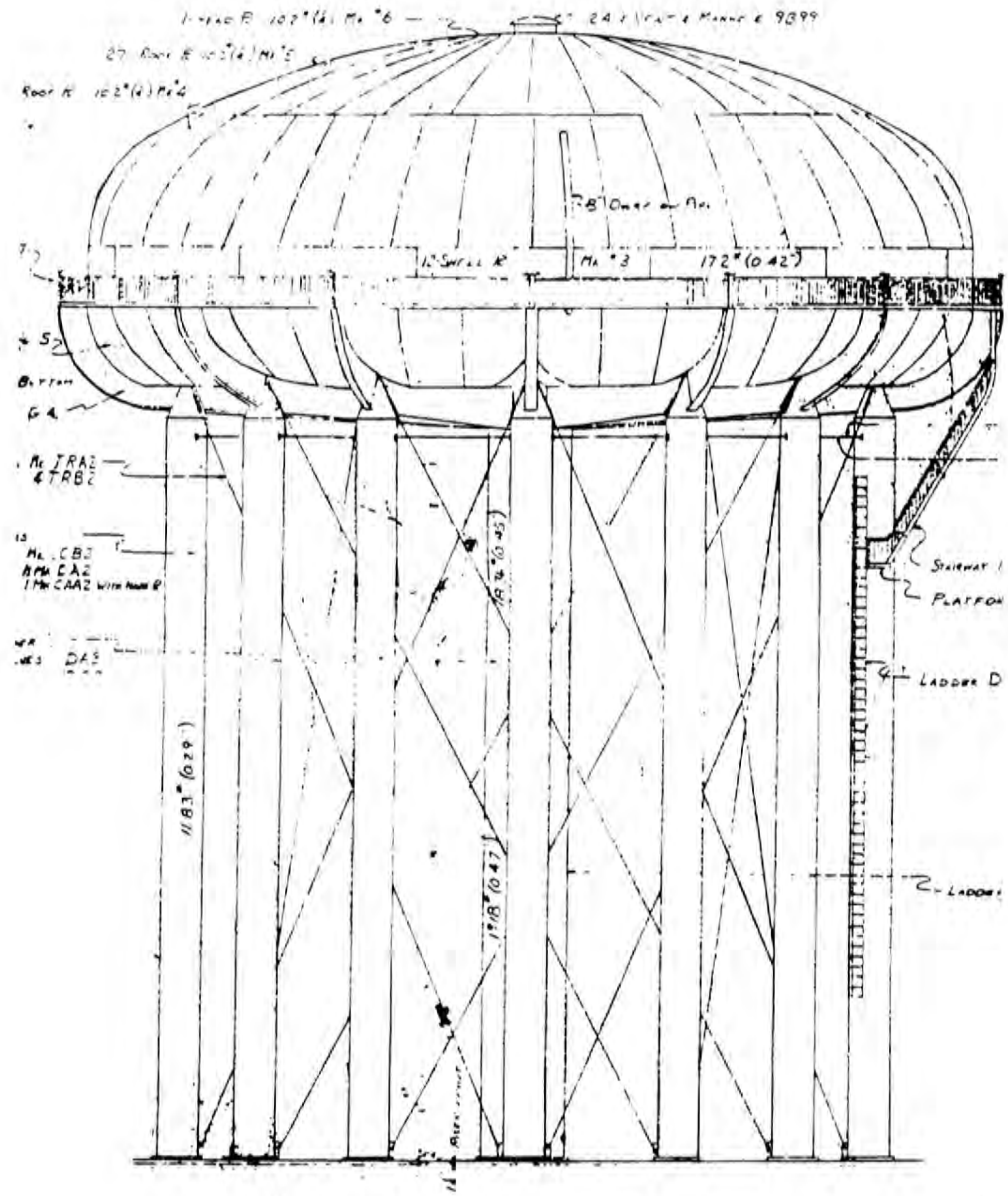
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	1.10	Minor yielding
Riser (tension in bending)	0.73	Adequate
Riser (compression)	0.75	Adequate
Columns	<1.00	Adequate
Brace Connections	1.00	Adequate
Column Base Connections	<1.00	Adequate
Radial Girder	0.81	Adequate
Soil Under Columns	<0.80 (allowable)	Adequate
Soil Under Riser	<0.55 (allowable)	Adequate

Elevated Tank Status: **Operable**

Table 14.4-3

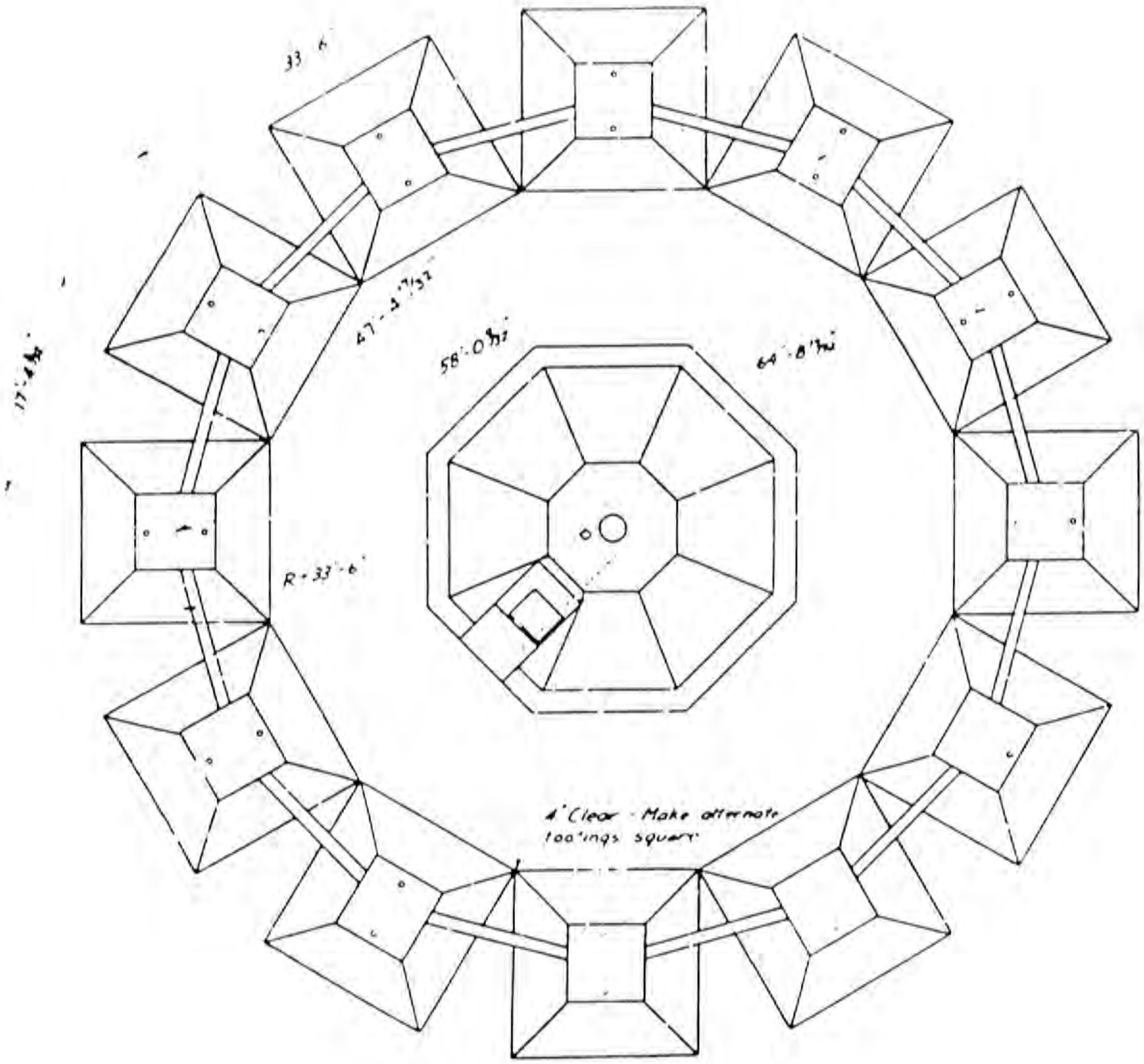
Magnolia Bluff Elevated Water Storage Tank (High Priority)
Design Base Shear Comparison

1988 UBC Code	1,743 kips
Class I Earthquake	694 kips
Class II Earthquake	430 kips



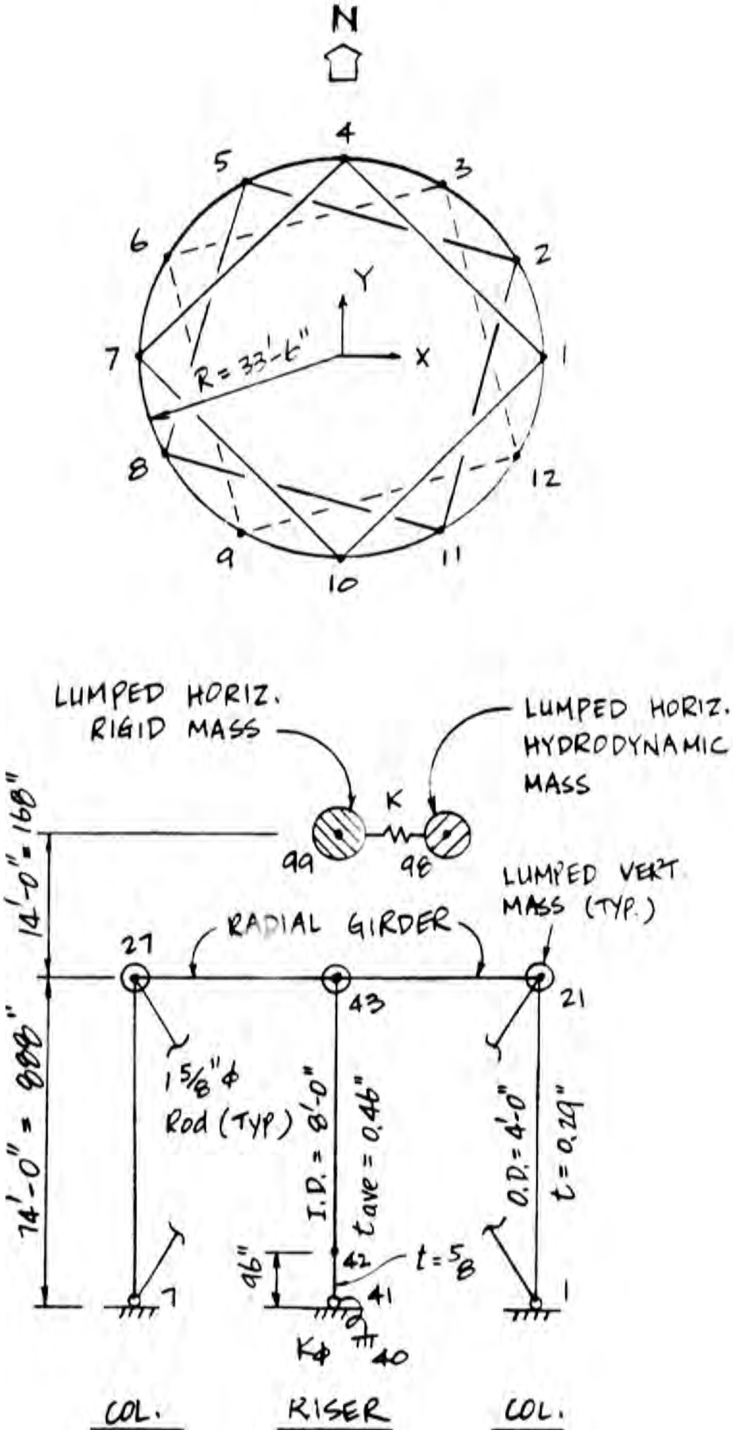
Magnolia Bluff Elevated Water Storage Tank
Elevation View

Figure 14.4-1



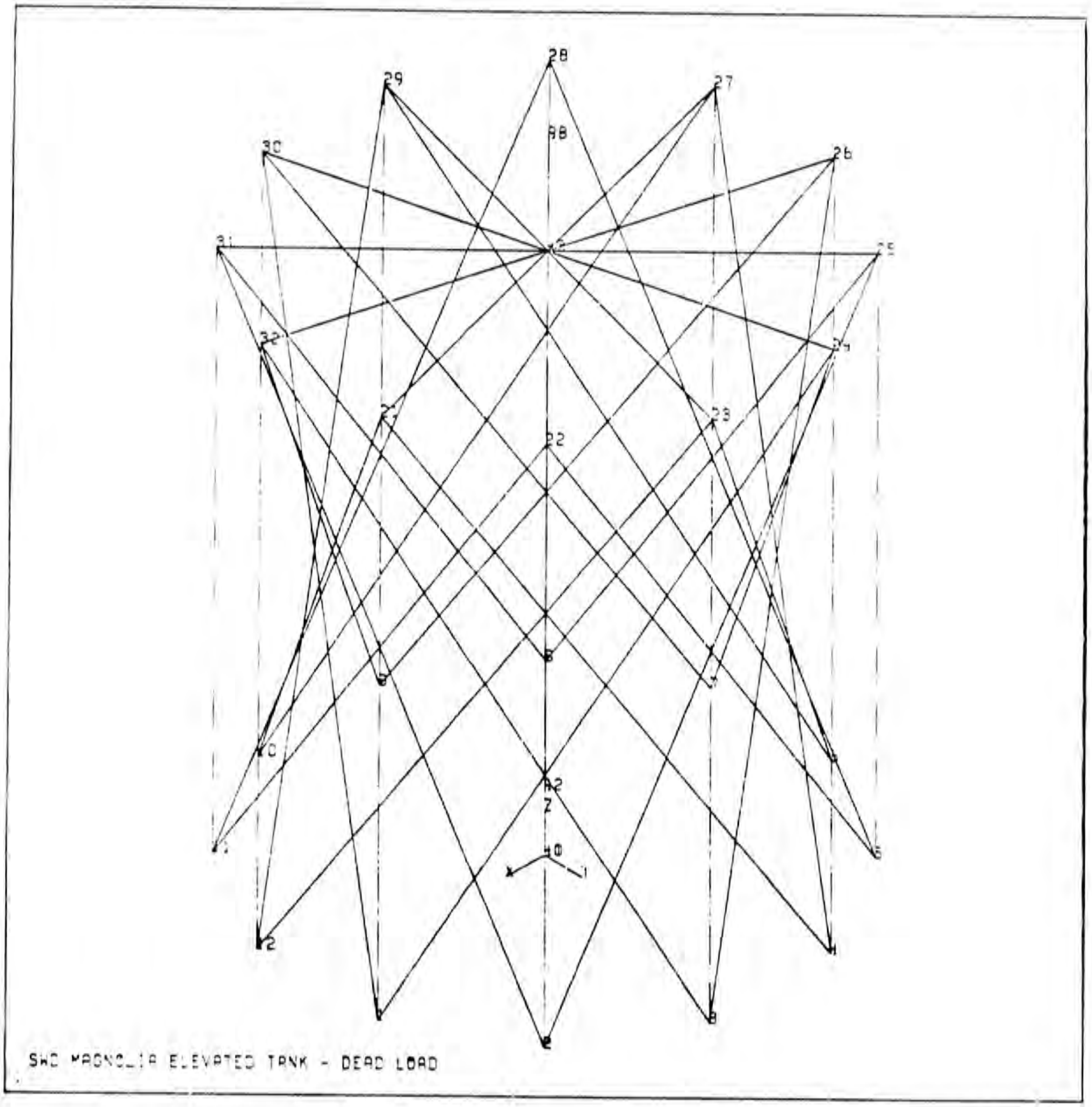
Magnolia Bluff Elevated Water Storage Tank
Plan View

Figure 14.4-2



Magnolia Bluff Elevated Water Storage Tank
Mathematical Model

Figure 14.4-3



Magnolia Bluff Elevated Water Storage Tank
Computer Plot

Figure 14.4-4

14.5 Maple Leaf

14.5.1 Facility Description

The Maple Leaf elevated water storage tank, built in 1949, has a capacity of 1,000,000 gallons. The structure is shown in Figures 14.5-1 to 14.5-4. The elevated tank shell is supported at its equator by ten (10) W14 X 119 braced columns with a cross sectional area of approximately 35 sq. in. The back-to-back angle braces have a cross section of 8 sq. in. There is a set of seven (7) unbraced interior columns and a central riser with a diameter of 120 in. and a cross section of approximately 166 sq. in.

The tank wall is a cylindrical shell with a stiffening flange to which the exterior columns are connected. The tops of the interior columns frame into radial girders. The columns are supported by spread footings with no tie beams.

The structure has a diameter of approximately 84 ft., and the center of gravity for the structure and contents is located at 77 ft. above grade level.

14.5.2 Analysis

The computer program ANSYS was used for analysis of the tank. Using this program, a three dimensional model of the tank was developed as shown in figure 14.5-3. The tank shell connection at the top of the columns was modeled by coupling the horizontal displacements at the top of the columns, and using an equivalent ring beam to restrain the rotations of the exterior columns and radial beams to restrain the rotations of the interior columns.

The connections at the base of each column are modeled as hinges. The riser has eight (8) anchor bolts and a spread footing. Rotational springs were modeled at the base of the riser.

Braces were modeled as capable of withstanding compression loads.

The water content was modeled using Housner's method. The weight of the water is distributed to the columns and riser in accordance to their tributary area.

14.5.3 Analysis Results

The fundamental period of vibration of the tank is 0.85 seconds, and the fluid sloshing period is 6 seconds. This places the tank period in the descending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to lowering the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 14.5.1 and 14.5.2. In our opinion the structure will not remain operable after the Class I earthquake. Connection failures and buckling of the braces are brittle failure modes which contribute to loss of stability.

The structure will remain operable after the Class II earthquake. Moderate soil over-stress will take place resulting in lower effective loads in the structure.

Table 14.5-3 shows a comparison of design base shears using the 1988 UBC Code and the Class I and the Class II earthquakes.

14.5.4 Upgrade Recommendations

a) This structure requires increased lateral strength. The cost effective way to achieve this is by introducing braces with a cross-sectional area of 4 sq. in. between the interior columns, using the same pattern as for the exterior columns.

b) Install new tie beams to connect the exterior column footings.

c) Install an additional 5.6 sq. in. of anchorage for each of the exterior columns.

d) Strengthen connections for existing braces. The connection capacities should exceed the brace capacities.

The estimated cost of upgrading the Maple Leaf elevated water storage tank, as shown in Figure 14.10-2, is $428,500.

Table 14.5-1

Maple Leaf Elevated Water Storage Tank (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	1.06	Minor yielding and buckling
Riser (tension in bending)	0.70	Adequate
Riser (compression)	0.84	Adequate
Interior Column	0.79	Adequate
Exterior Column	0.95	Adequate
Radial Beams	0.74	Adequate
Brace Connections	1.35	Rivets shear
Riser Base Connection	>1.00	Anchors yield
Column Base Connection	6.65	Anchors rupture, foundation uplifts
Soil Under Exterior Columns	1.11 (allowable)	Minor overstress
Soil Under Interior Columns	1.22 (allowable)	Overstress
Soil Under Riser	1.23 (allowable)	Rotation of riser footing expected

Elevated Tank Status: **Not Operable**

Estimated Probability of Failure: **80%**

Table 14.5-2

Maple Leaf Elevated Water Storage Tank
Analysis Results

Class II Earthquake

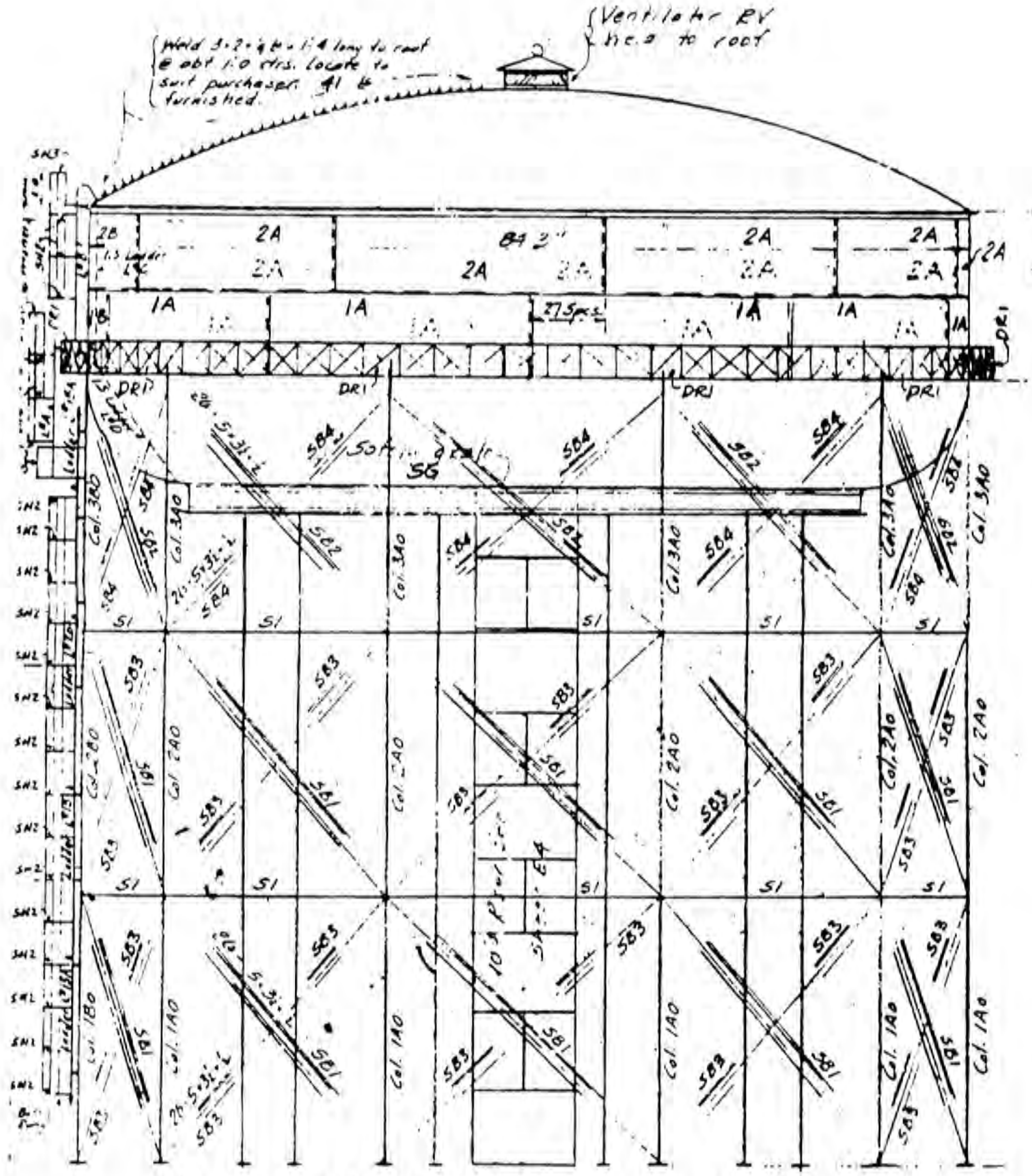
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	0.7	Adequate
Riser (tension in bending)	<0.84	Adequate
Interior Column	<0.79	Adequate
Exterior Column	<0.95	Adequate
Radial Beams	<0.74	Adequate
Brace Connections	1.00	Adequate
Riser Base Connection	>1.00	Adequate
Column Base Connection	3.50	Anchor yield
Soil Under Columns	0.88 (allowable)	Adequate
Soil Under Riser	>1.00 (allowable)	Rotation of riser footing expected

Elevated Tank Status: **Operable**

Table 14.5-3

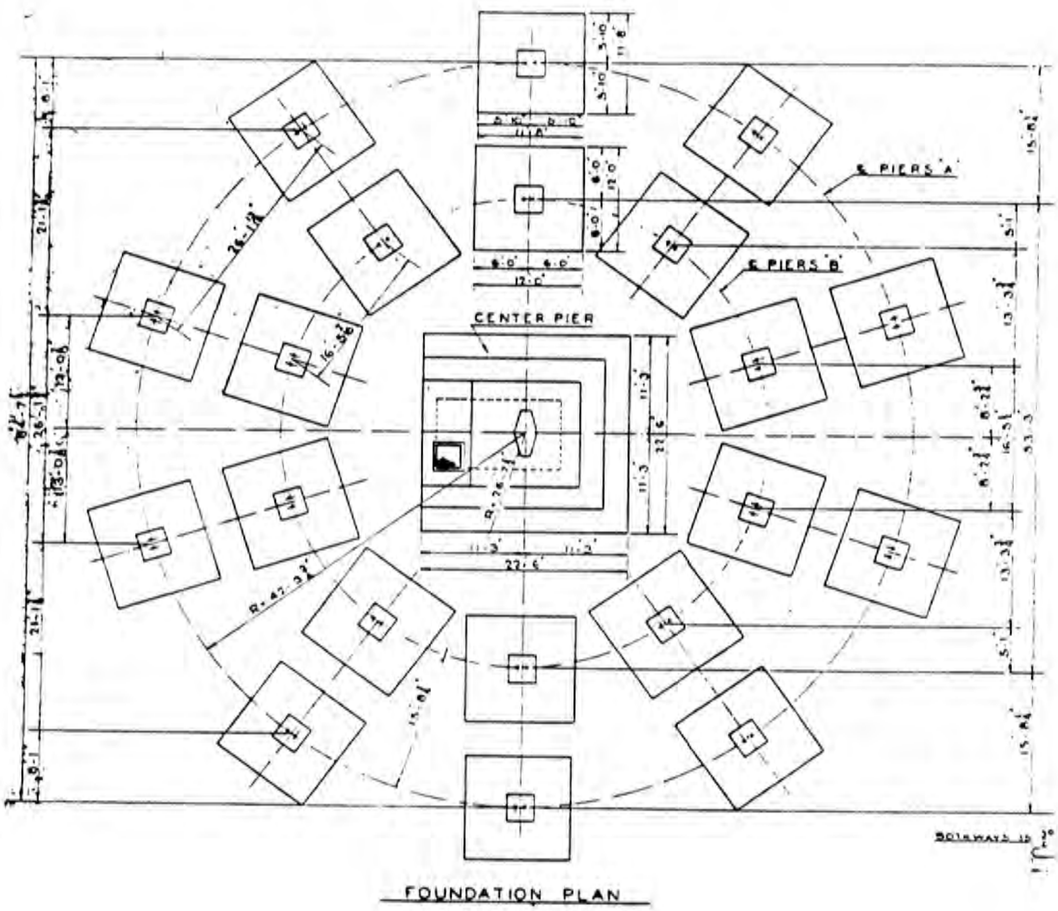
Maple Leaf Elevated Water Storage Tank (High Priority)
Design Base Shear Comparison

1988 UBC Code	1,969 kips
Class I Earthquake	1,807 kips
Class II Earthquake	1,120 kips



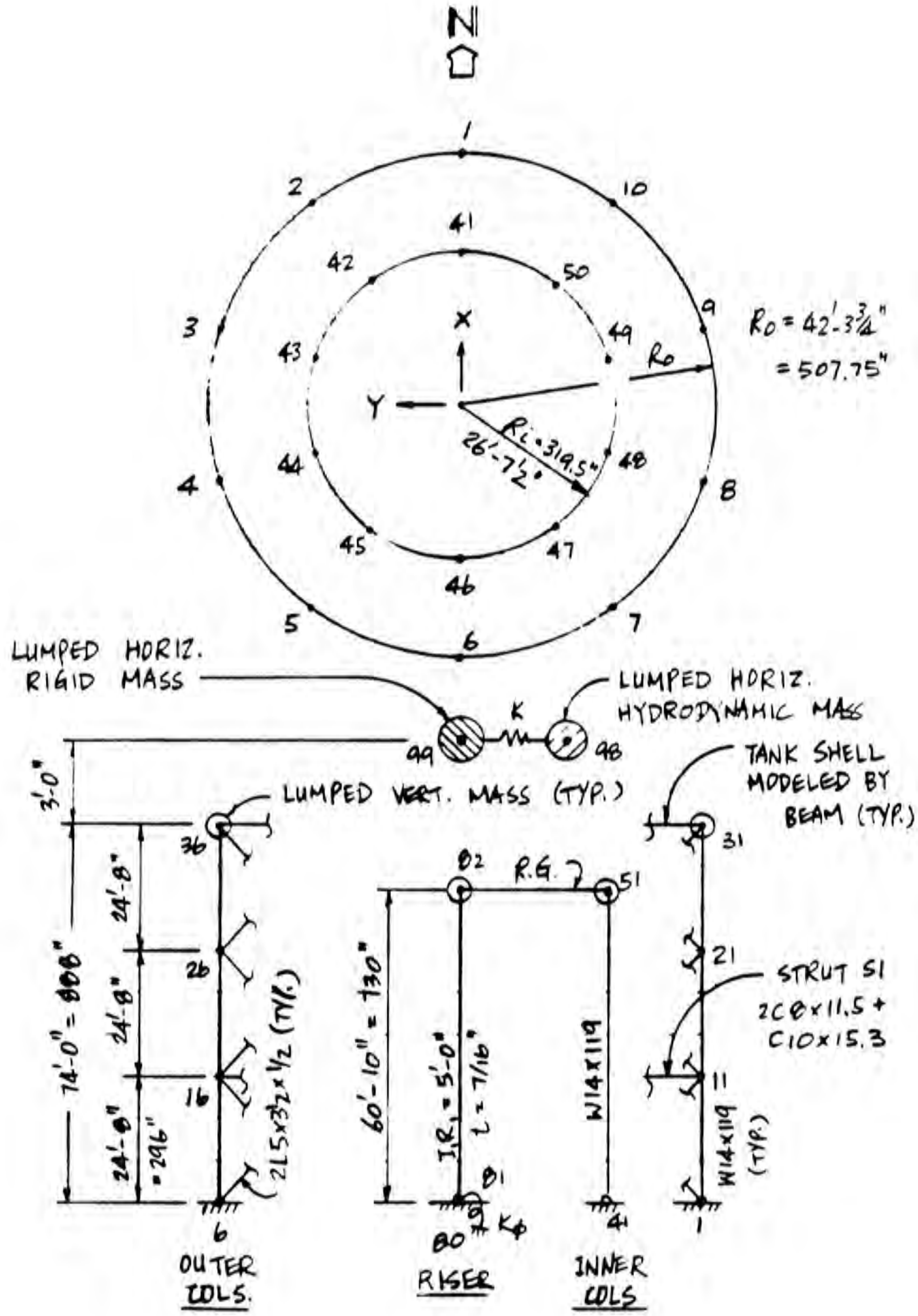
Maple Leaf Elevated Water Storage Tank
Elevation View

Figure 14.5-1



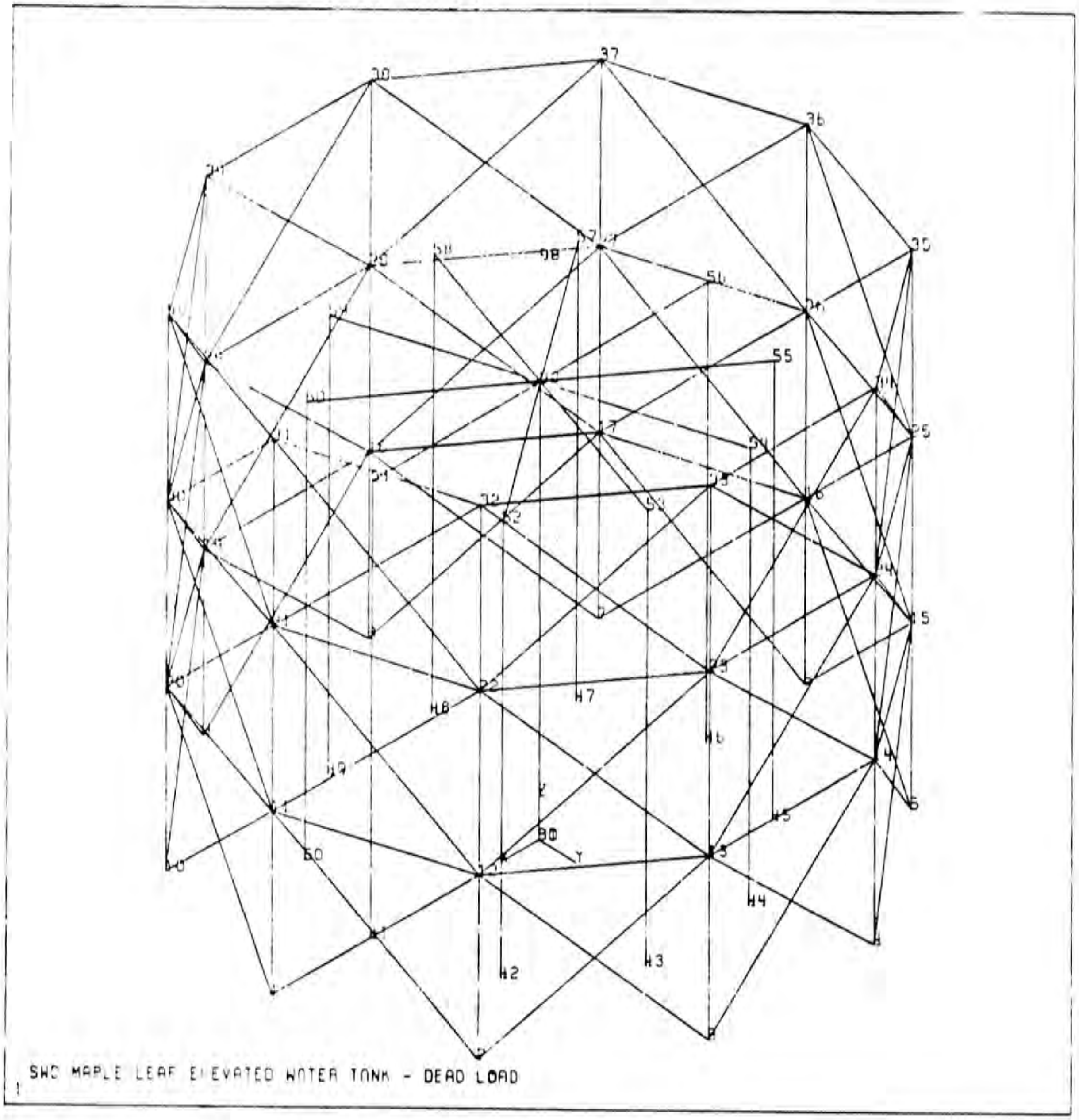
Maple Leaf Elevated Water Storage Tank
Plan View

Figure 14.5-2



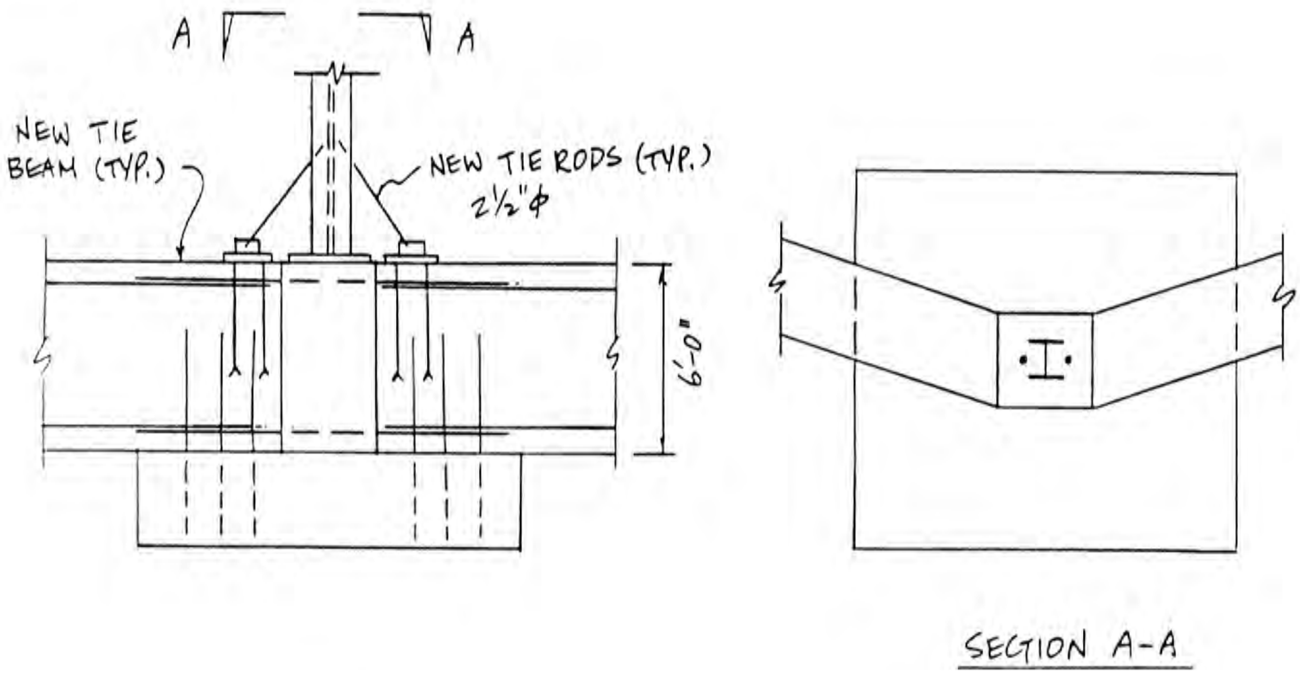
Maple Leaf Elevated Water Storage Tank
Mathematical Model

Figure 14.5-3



Maple Leaf Elevated Water Storage Tank
Computer Plot

Figure 14.5-4



Maple Leaf Elevated Water Storage Tank
Conceptual Foundation Upgrades

Figure 14.5-5

14.6 S.W. Myrtle #1

14.6.1 Facility Description

The S.W. Myrtle #1 elevated water storage tank, built in 1919, has a capacity of 500,000 gallons. The structure is shown in Figures 14.6-1 to 14.6-4. The elevated tank shell is supported by eight (8) C 15 x 55 laced columns with a cross sectional area of approximately 32 sq. in. The rod braces have a cross section of 1-1/2 sq. in. There is a central riser with a diameter of 60 in. and a cross section of approximately 47 sq. in.

The tank wall is a cylindrical shell with a stiffening flange to which the columns are connected. The columns are supported by spread footings with no tie beams.

The structure has a diameter of approximately 46 ft., and the center of gravity for the structure and contents is located at 48 ft. above grade level. No structural details were available for the column and riser connections.

14.6.2 Analysis

The computer program ANSYS was used for analysis of the tank. Using this program, a three dimensional model of the tank was developed as shown in Figure 14.6-3. The tank shell connection at the top of the columns was modeled by coupling the horizontal displacements at the top of the columns, and using an equivalent ring beam to restrain the rotations. The connections at the base of each column and the riser are modeled as hinges.

Braces are not capable of withstanding compression loads. Only half of the brace area was included in the model. This accounts for the braces not being effective in compression.

Column stresses were based on the axial reaction at the base of the footings. In this manner, the column stresses are not artificially reduced by the braces taking some axial load.

The water content was modeled using Housner's method. The weight of the water was distributed to the columns and riser in accordance to their tributary area.

14.6.3 Analysis Results

The fundamental period of vibration of the tank is 2.1 seconds, and the fluid sloshing period is 3.3 seconds. This places the tank period in the descending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to lowering the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 14.6-1 and 14.6-2. In our opinion the structure will not remain operable after either the Class I or the Class II earthquakes. Connection failures and buckling of the columns are brittle failure modes which contribute to loss of stability. In addition, the braces are severely overstressed.

Table 14.6-3 shows a comparison of design base shears using the 1988 UBC Code and the Class I and the Class II earthquakes.

14.6.4 Upgrade Recommendations

This structure requires increased lateral strength. It should be noted that if the columns and braces are strengthened beyond a certain limit, problems will arise at the foundation level (soil overstress). A cost-effective strategy is to limit the level of upgrade by allowing the braces to undergo some yielding.

These upgrade recommendations are:

a) Replace the lacing between the C sections in the columns with 3/4 in. plates.

b) Strengthen or replace the existing rod braces with 2-in. diameter rods.

c) Strengthen the connections accordingly, such that the connection capacities exceed the brace capacities.

It is also recommended that an investigation be conducted to verify that the soil bearing capacity is greater than the 5.33 ksf used in the analysis.

The estimated cost of upgrading the S.W. Myrtle #1 elevated water storage tank, as shown in Table 14.10-2, is $433,300.

Table 14.6-1

S.W. Myrtle #1 Elevated Water Storage Tank (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	4.60	Braces yield and possibly rupture
Riser Tension	0.26	Adequate
Riser Compression	0.39	Adequate
Columns Tension	2.20	Yielding
Columns Compression	2.40	Buckling
Brace Connections	1.02	Adequate
Soil Under Columns	1.00 (allowable)	Adequate
Soil Under Riser	1.50 (allowable)	Rotation of riser footing expected

Elevated Tank Status: **Not Operable**

Estimated Probability of Failure: **90%**

Table 14.6-2

S.W. Myrtle #1 Elevated Water Storage Tank (High Priority) Analysis Results

Class II Earthquake

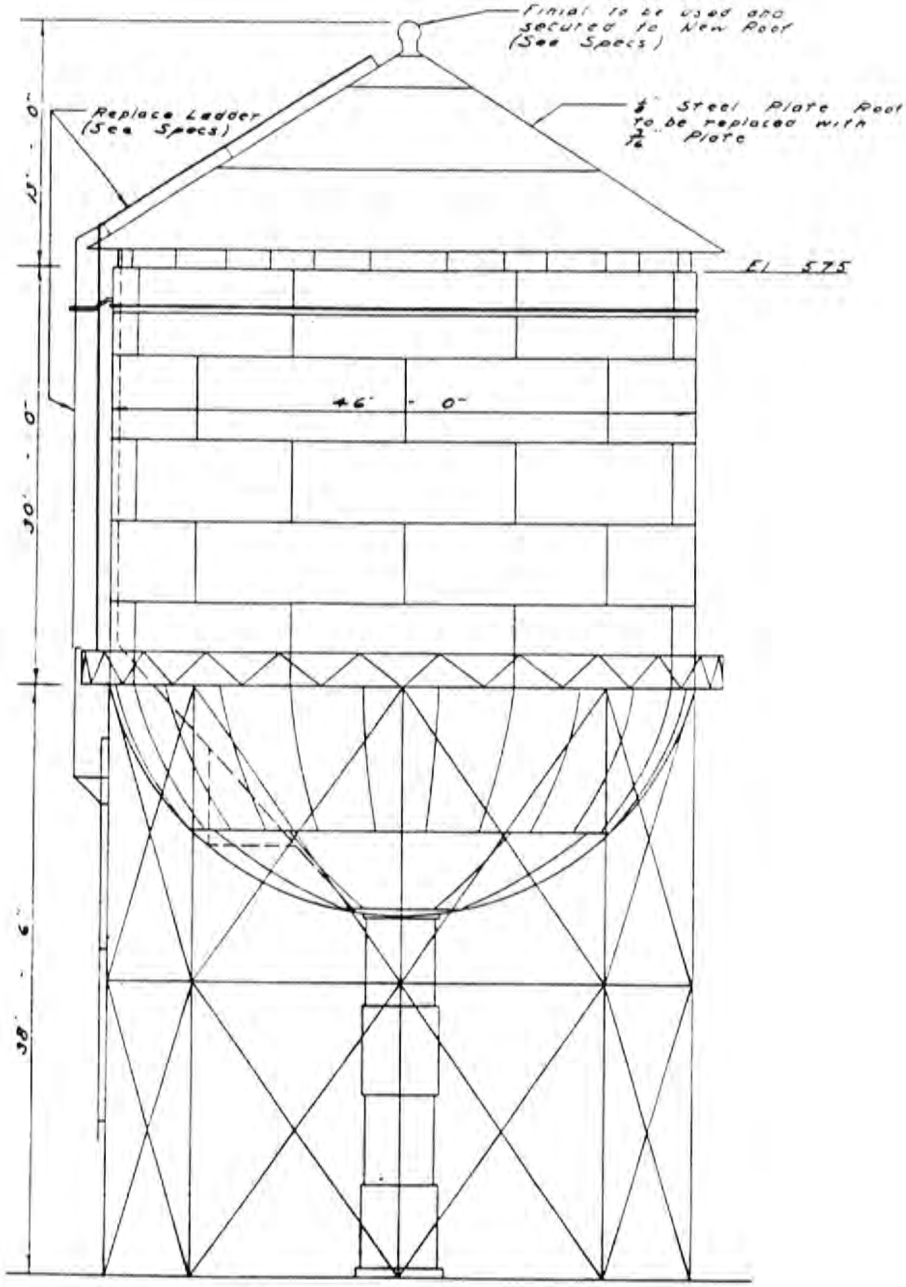
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	2.90	Braces yield
Riser	<0.39	Adequate
Column Tension	1.72	Yielding
Column Compression	1.72	Buckling
Brace Connections	<1.00	Adequate
Soil Under Columns	<1.00 (allowable)	Adequate
Soil Under Riser	1.41 (allowable)	Rotation of riser footing expected

Elevated Tank Status: **Not Operable**

Table 14.6-3

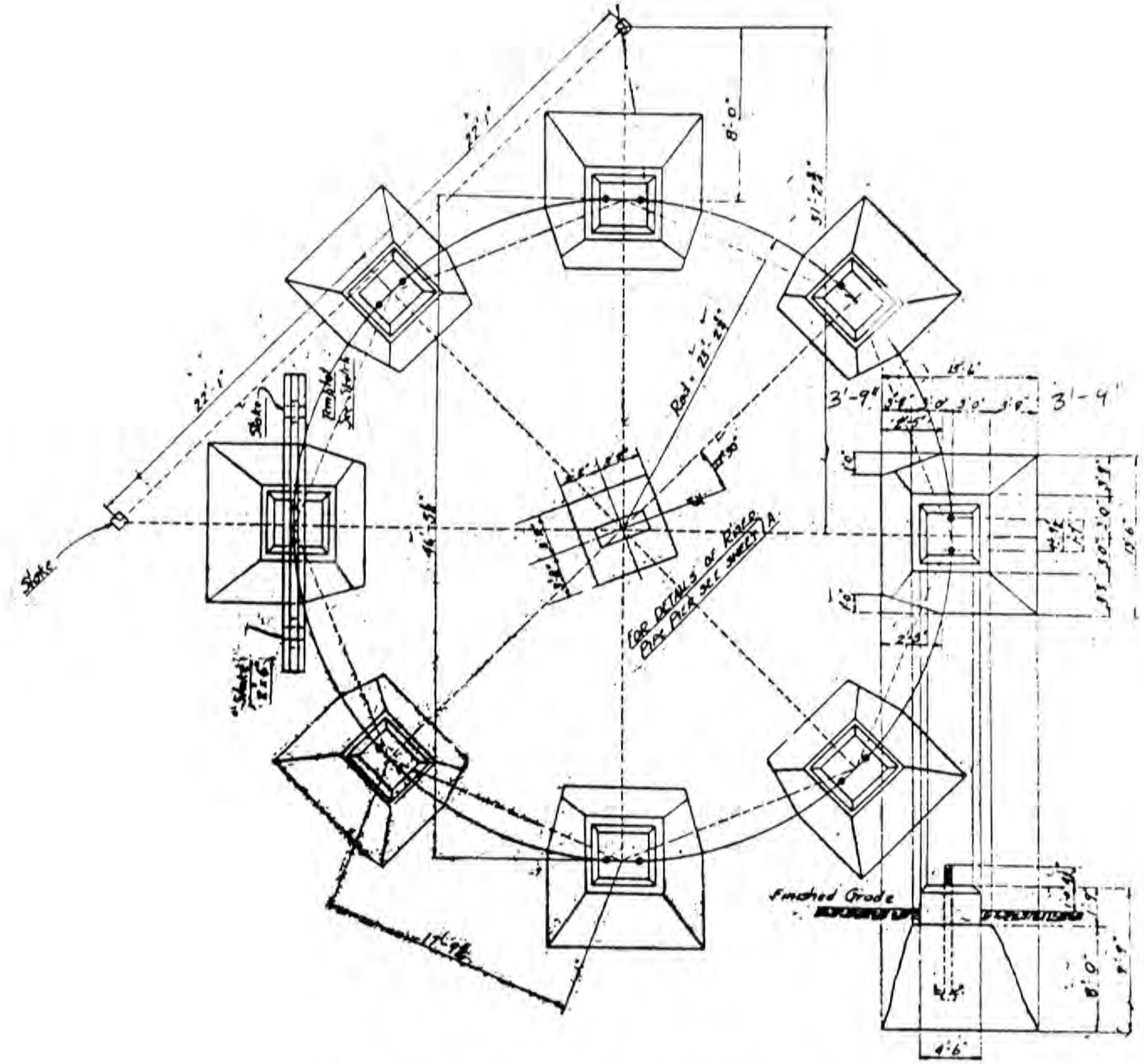
**S.W. Myrtle #1 Elevated Water Storage Tank (High Priority)
Design Base Shear Comparison**

1988 UBC Code	838 kips
Class I Earthquake	747 kips
Class II Earthquake	463 kips



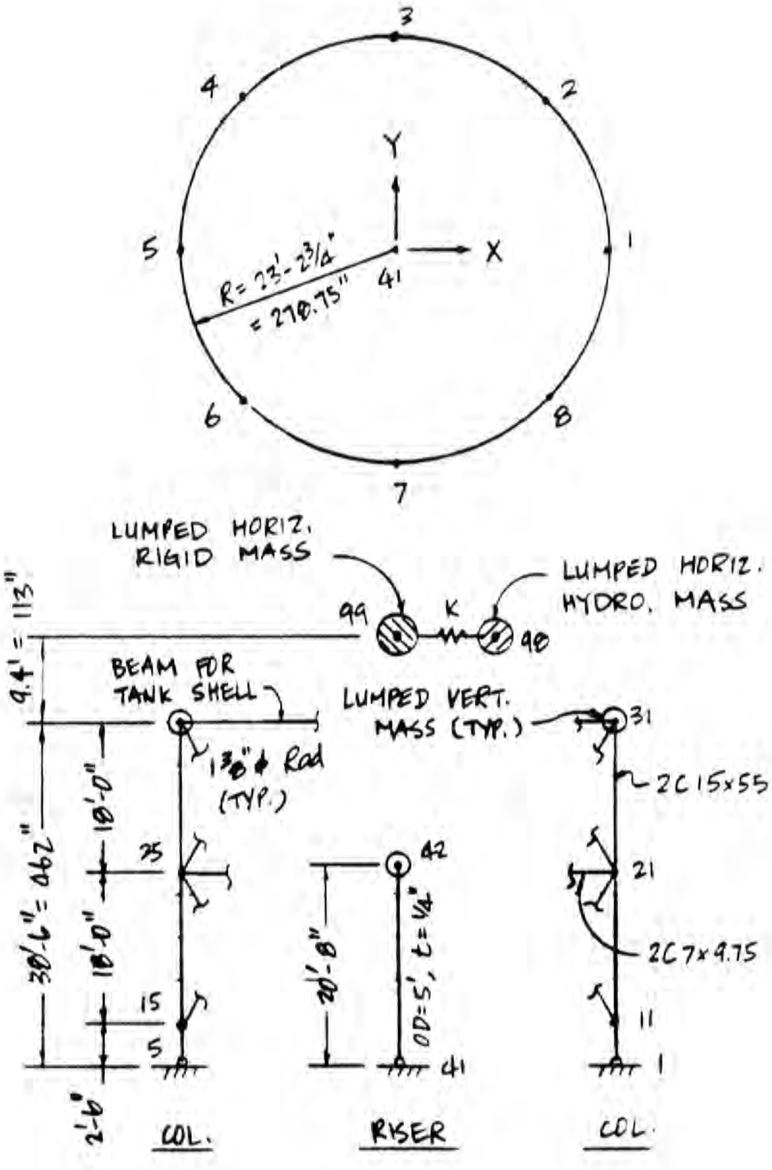
S.W. Myrtle #1 Elevated Water Storage Tank
Elevation View

Figure 14.6-1



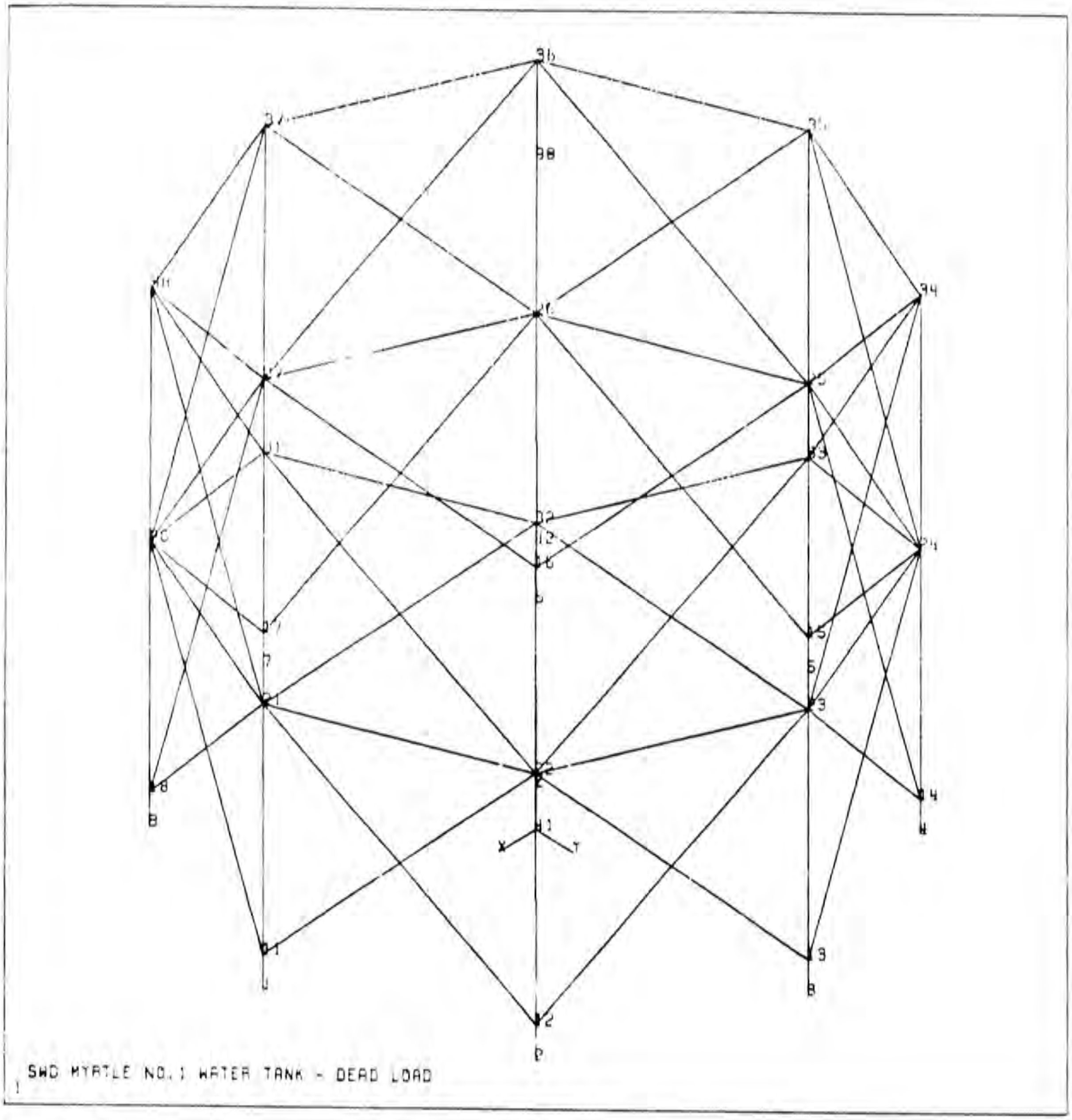
S.W. Myrtle #1 Elevated Water Storage Tank
Plan View

Figure 14.6-2



S.W. Myrtle #1 Elevated Water Storage Tank
Mathematical Model

Figure 14.6-3



S.W. Myrtle #1 Elevated Water Storage Tank
Computer Plot

Figure 14.6-4

14.7 S.W. Myrtle #2

14.7.1 Facility Description

The S.W. Myrtle #2 elevated water storage tank, built in 1946, has a capacity of 1,000,000 gallons. The structure is shown in Figures 14.7-1 to 14.7-4. The elevated tank shell is supported at its equator by ten (10) W14 X 103 braced columns with a cross sectional area of approximately 30 sq. in. The rod braces have a cross section of 1 sq. in. There is a set of ten (10) braced interior columns with 1-in. diameter rod braces and a central riser with a diameter of 120 in. and a cross section of approximately 166 sq. in.

The tank wall is a cylindrical shell with a stiffening flange to which the exterior columns are connected. The tops of the interior columns frame into radial girders. The columns are supported by spread footings with no tie beams.

The structure has a diameter of approximately 84 ft., and the center of gravity for the structure and contents is located at 56 ft. above grade level.

14.7.2 Analysis

The computer program ANSYS was used for analysis of the tank. Using this program, a three dimensional model of the tank was developed as shown in Figure 14.7-3. The tank shell connection at the top of the columns was modeled by coupling the horizontal displacements at the top of the columns, and using an equivalent ring beam to restrain the rotations of the exterior columns and radial beams to restrain the rotations of the interior columns (see Figure 14.7-3).

The connections at the base of each column are modeled as hinges. The riser has eight anchor bolts and a spread footing. Rotational springs were modeled at the base of the riser.

Braces are not capable of withstanding compression loads. Only half of the brace area was included in the model. This accounts for the braces not being effective in compression.

Column stresses were based on the axial reaction at the base of the footings. In this manner the column stresses are not artificially reduced by the braces taking some axial load.

The water content was modeled using Housner's method. The weight of the water is distributed to the columns and riser in accordance to their tributary area.

14.7.3 Analysis Results

The fundamental period of vibration of the tank is 0.99 seconds, and the fluid sloshing period is 5.9 seconds. This places the tank period in the descending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to lowering the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 14.7-1 and 14.7-2. In our opinion the structure will not remain operable after either the Class I or the Class II earthquakes. Connection failures and buckling of the columns are brittle failure modes contributing to loss of stability. In addition, the braces are severely overstressed.

Table 14.7-3 shows a comparison of design base shears using the 1988 UBC Code and the Class I and Class II earthquakes.

14.7.4 Upgrade Recommendations

This structure requires increased lateral strength. It should be noted that if the columns and braces are strengthened beyond a certain limit, problems will arise at the foundation level (soil overstress). A cost-effective strategy is to limit the level of upgrade by allowing the braces to undergo some yielding.

These upgrade recommendations are:

a) Increase the cross sectional area of the interior columns by 15 sq. in. each.

b) Strengthen or replace the existing rod braces with 1-1/2 in. diameter rods for the interior columns and 1-3/4 in. diameter rods for the exterior columns.

c) Strengthen the connections accordingly, such that the connection capacities exceed the brace capacities.

It is also recommended to verify that the soil bearing capacity is greater than the 5.33 ksf used in the analysis.

The estimated cost of upgrading the S.W. Myrtle #2 elevated water storage tank, as shown in Table 14.10-2, is $585,400.

Table 14.7-1

S.W. Myrtle #2 Elevated Water Storage Tank
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	3.20	Yielding
Riser (tension in bending)	1.26	Yielding
Riser (compression)	1.70	Buckling
Interior Column	1.50	Buckling and yielding
Exterior Column	1.44	Buckling and yielding
Radial Beams	1.23	Minor yielding
Brace Connections	1.07	Not adequate
Riser Base Connection	2.30	Anchors yield
Soil Under Exterior Columns	0.56 (allowable)	Adequate
Soil Under Interior Columns	1.37 (allowable)	Rotation of column footing expected
Soil Under Riser	1.23 (allowable)	Rotation of riser footing expected

Elevated Tank Status: **Not Operable**

Estimated Probability of Failure: **65%**

Table 14.7-2

S.W. Myrtle #2 Elevated Water Storage Tank (High Priority)
Analysis Results

Class II Earthquake

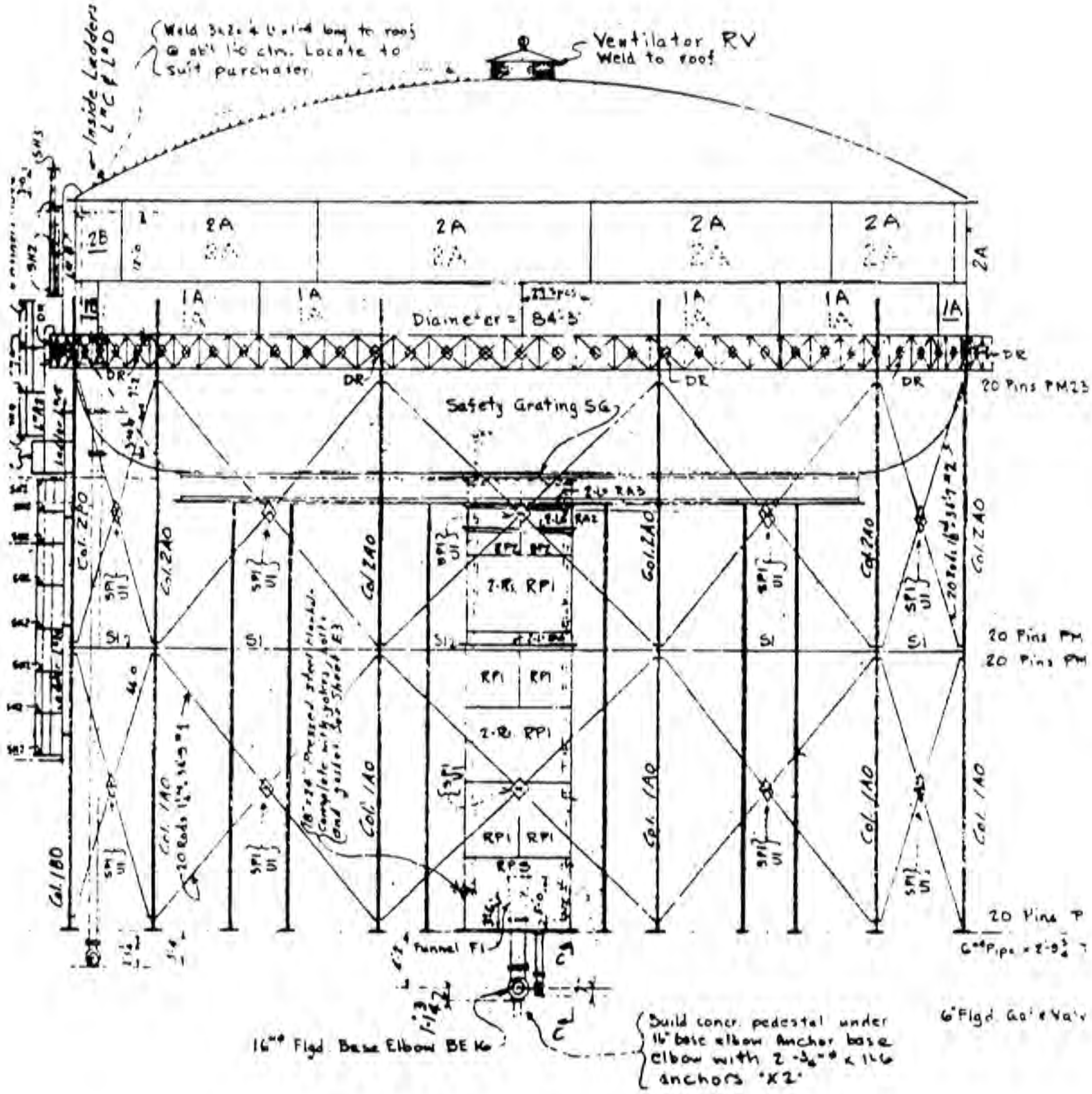
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	2.0	Yielding
Riser (compression in bending)	<1.26	Buckling
Interior Column	1.38	Buckling and yielding
Exterior Column	1.00	Adequate
Radial Beams	0.90	Adequate
Brace Connections	1.00	Adequate
Riser Base Connection	1.95	Anchors yield
Soil Under Columns	<1.37 (allowable)	Rotation of column footing expected
Soil Under Riser	<1.23 (allowable)	Rotation of riser footing

Elevated Tank Status: **Not Operable**

Table 14.7-3

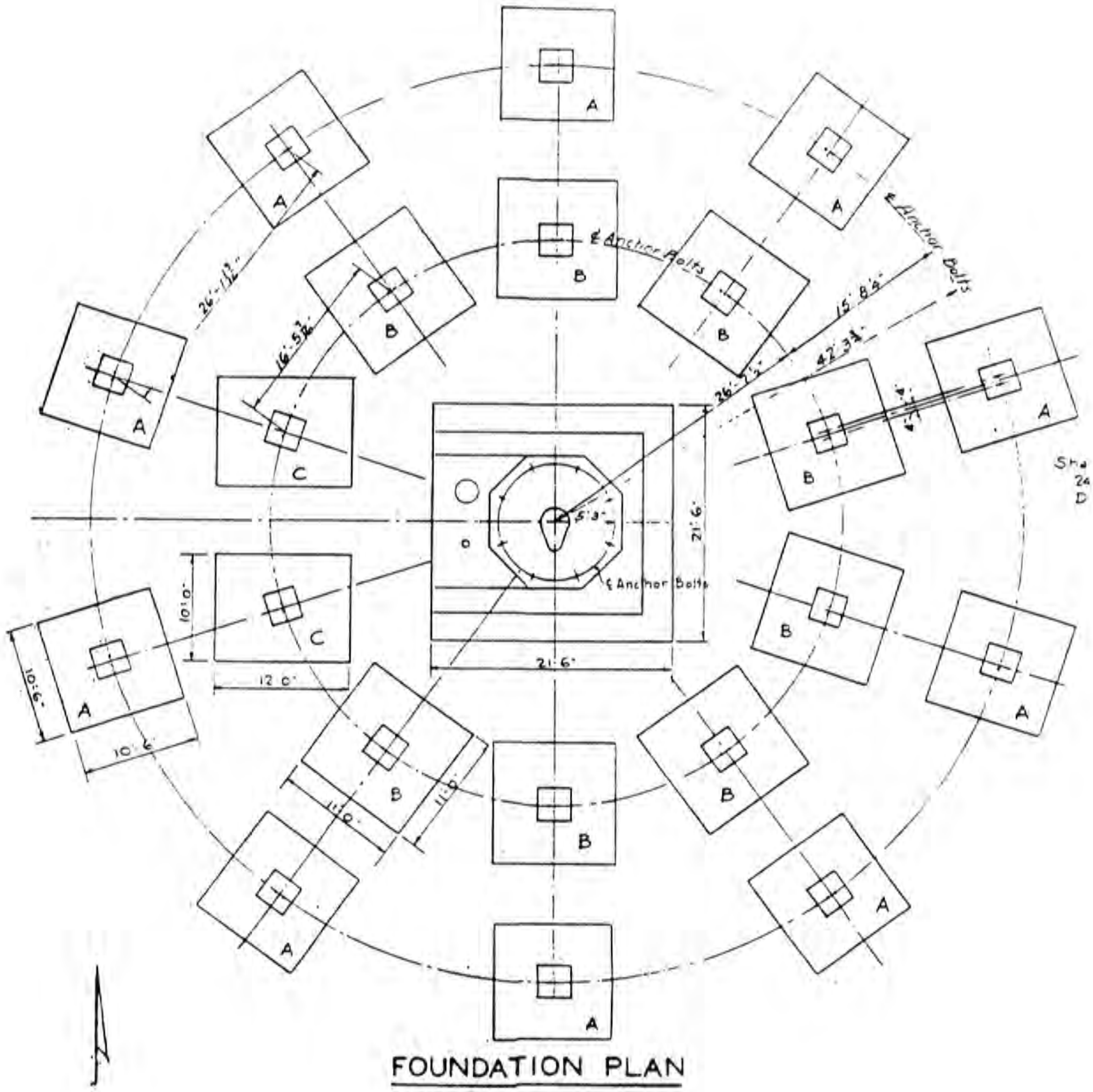
**S.W. Myrtle #2 Elevated Water Storage Tank (High Priority)
Design Base Shear Comparison**

1988 UBC Code	1,727 kips
Class I Earthquake	1,363 kips
Class II Earthquake	845 kips



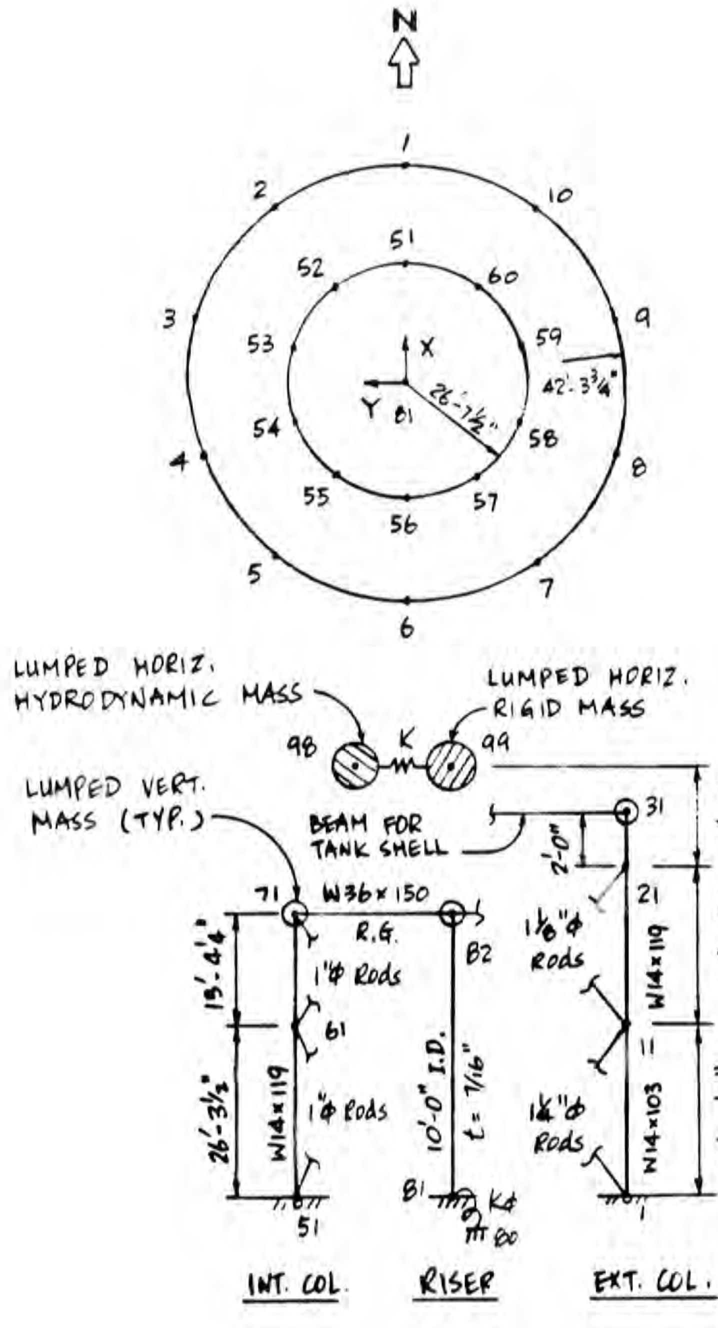
S.W. Myrtle #2 Elevated Water Storage Tank
Elevation View

Figure 14.7-1



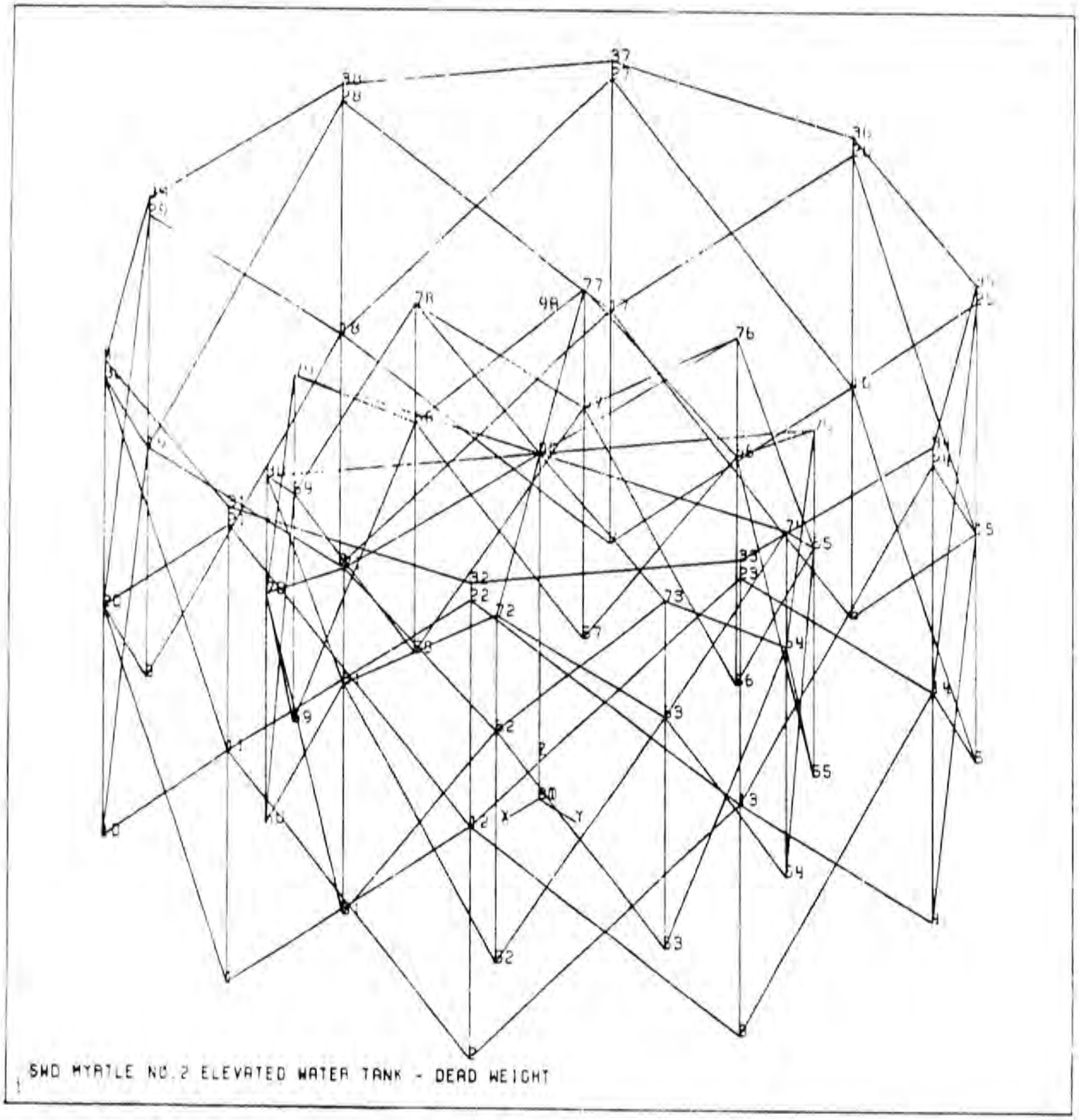
S.W. Myrtle #2 Elevated Water Storage Tank Plan View

Figure 14.7-2



S.W. Myrtle #2 Elevated Water Storage Tank Mathematical Model

Figure 14.7-3



S.W. Myrtle #2 Elevated Water Storage Tank
Computer Plot

Figure 14.7-4

14.8 Richmond Highlands #1

14.8.1 Facility Description

The Richmond Highlands #1 elevated water storage tank, built in 1954, has a capacity of 1,000,000 gallons. The structure is shown in Figures 14.8-1 to 14.8-4. The elevated tank shell is supported by radial girders resting on twelve (12) braced columns with a diameter of 48 in. and a cross sectional area of approximately 43 sq. in. The braces have a cross section of 7 sq. in. There is a central riser with a diameter of 96 in. and a cross section of approximately 139 sq. in. The columns are supported by spread footings with no tie beams.

The structure has a diameter of approximately 66 ft., and the center of gravity for the structure and contents is located at 75 ft. above grade level.

14.8.2 Analysis

The computer program ANSYS was used in the analysis. The three dimensional model is shown in Figure 14.8-3. The tank shell connection at the top of the columns was modeled by coupling the horizontal radial beams to the top of the columns, and using constraint equations to couple the horizontal displacements without further restraining the rotations. This assumption is justified because at those connections forces are transferred primarily by shear along the tangential surface of the tank. Along this tangential direction the tank shell is very rigid.

Only half of the brace area is included in the model. This approach accounts for the braces not being effective in compression.

Column stresses were computed on the basis of axial reactions at the base of the footings. In this manner the column stresses are not artificially reduced by the brace elements taking some axial load.

The connections at the base of each column are modeled as hinges. The riser has six (6) anchor bolts. Rotational springs were modeled at the base of the riser.

The water content was modeled using Housner's method. The weight of the water was distributed to the columns and riser in accordance to their tributary area.

14.8.3 Analysis Results

The fundamental period of vibration of the tank is 0.94 seconds, and the fluid sloshing period is 4.9 seconds. This places the tank period in the descending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to lowering the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 14.8-1 and 14.8-2. In our opinion, the structure will not remain operable after either the Class I or the Class II earthquake. Yielding of the braces and yielding and buckling of the columns will contribute to instability of the structure.

A comparison of design base shears computed using the 1988 UBC Code and the Class I and the Class II earthquakes is provided in Table 4.4-3.

14.8.4 Upgrade Recommendations

This structure requires increased lateral strength. It should be noted that if the columns and braces are strengthened beyond a certain limit, problems will arise at the foundation level (soil overstress). A cost-effective strategy is to limit the level of upgrade by allowing the braces to undergo some yielding.

These recommended upgrades are:

a) Increase the cross sectional area of the columns using 3/16-in. plates.

b) Strengthen or replace the existing pair of rod braces with 2-3/4 in. diameter rods.

c) Strengthen the connections accordingly, such that the connections capacities exceed the brace capacities.

The estimated cost of upgrading the Richmond Highlands #1 elevated water storage tank, as shown in Table 14.10-2, is $491,200.

Table 14.8-1

Richmond Highlands #1 Elevated Water Storage Tank (High Priority) Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	1.60	Yielding
Riser (tension in bending)	0.95	Adequate
Riser (compression)	0.96	Adequate
Riser Base Connection	1.10	Anchors yield
Columns	1.34	Yielding and buckling
Brace Connections	0.29	Adequate, limited by braces yielding
Radial Girder	0.97	Adequate
Soil Under Columns	0.81 (allowable)	Limited by braces yielding
Soil Under Riser	0.20 (allowable)	Limited by anchors yielding

Elevated Tank Status: **Not Operable**

Estimated Probability of Failure: **70%**

Table 14.8-2

Richmond Highlands #1 Elevated Water Storage Tank (High Priority)
Analysis Results

Class II Earthquake

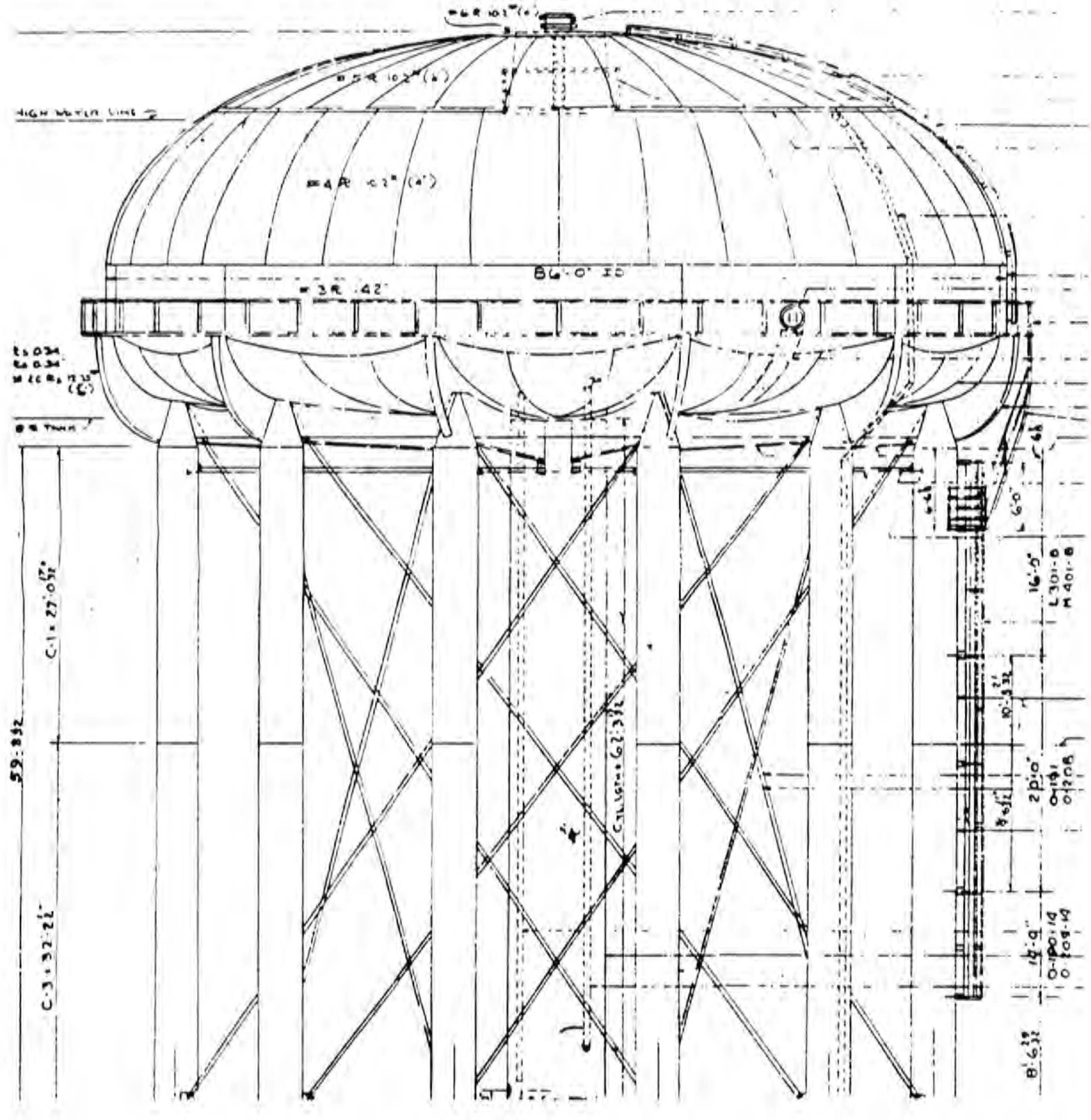
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	0.98	Adequate
Riser (tension in bending)	<0.95	Adequate
Riser (compression)	<0.95	Adequate
Columns	1.16	Yielding and buckling
Brace Connections	<0.29	Adequate
Radial Girder	0.81	Adequate
Soil Under Columns	<0.81 (allowable)	Adequate
Soil Under Riser	<0.20 (allowable)	Adequate

Elevated Tank Status: **Not Operable**

Table 14.8.3

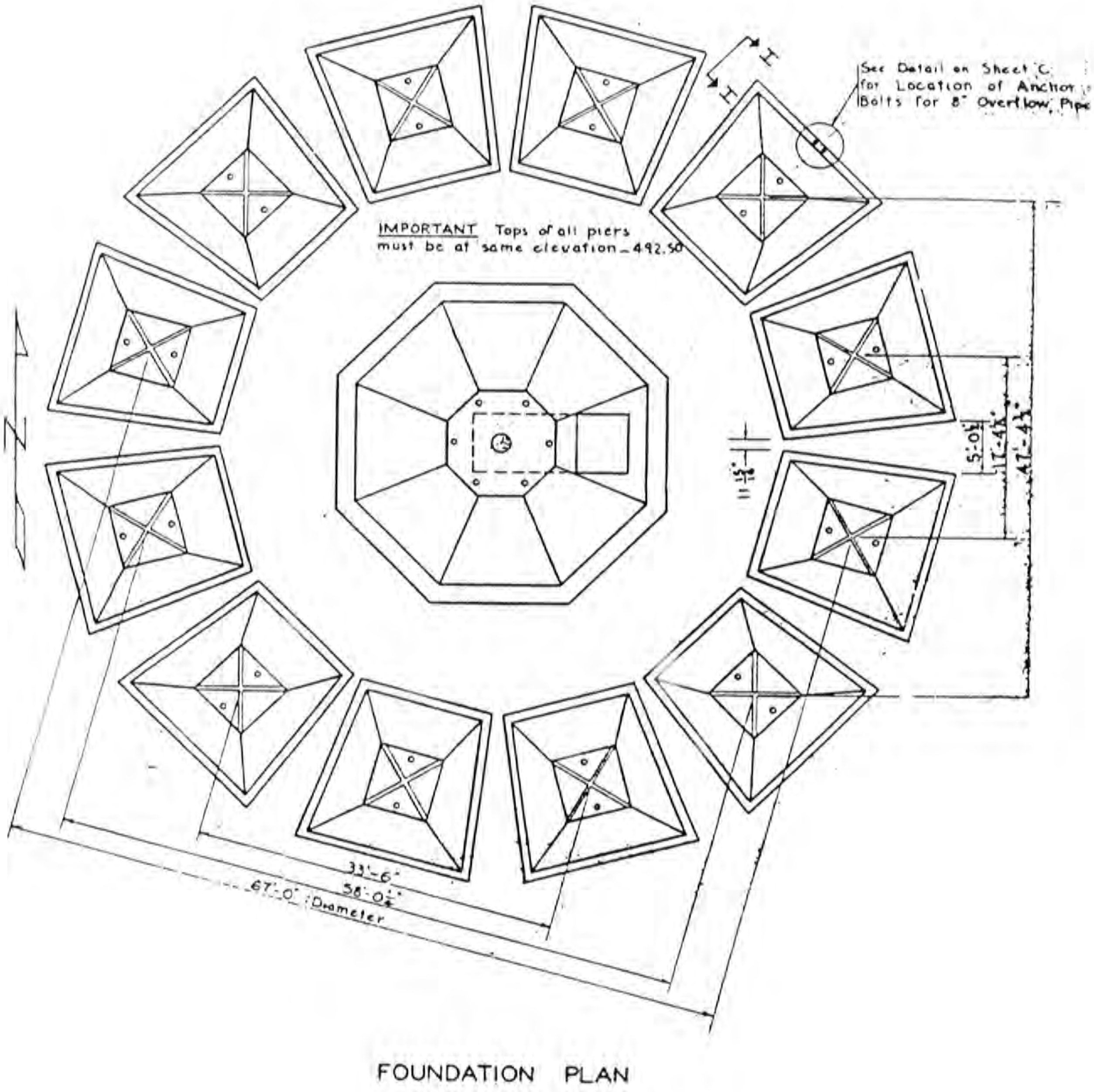
Richmond Highlands #1 Elevated Water Storage Tank (High Priority)
Design Base Shear Comparison

1988 UBC Code	1,774 kips
Class I Earthquake	1,613 kips
Class II Earthquake	1,000 kips



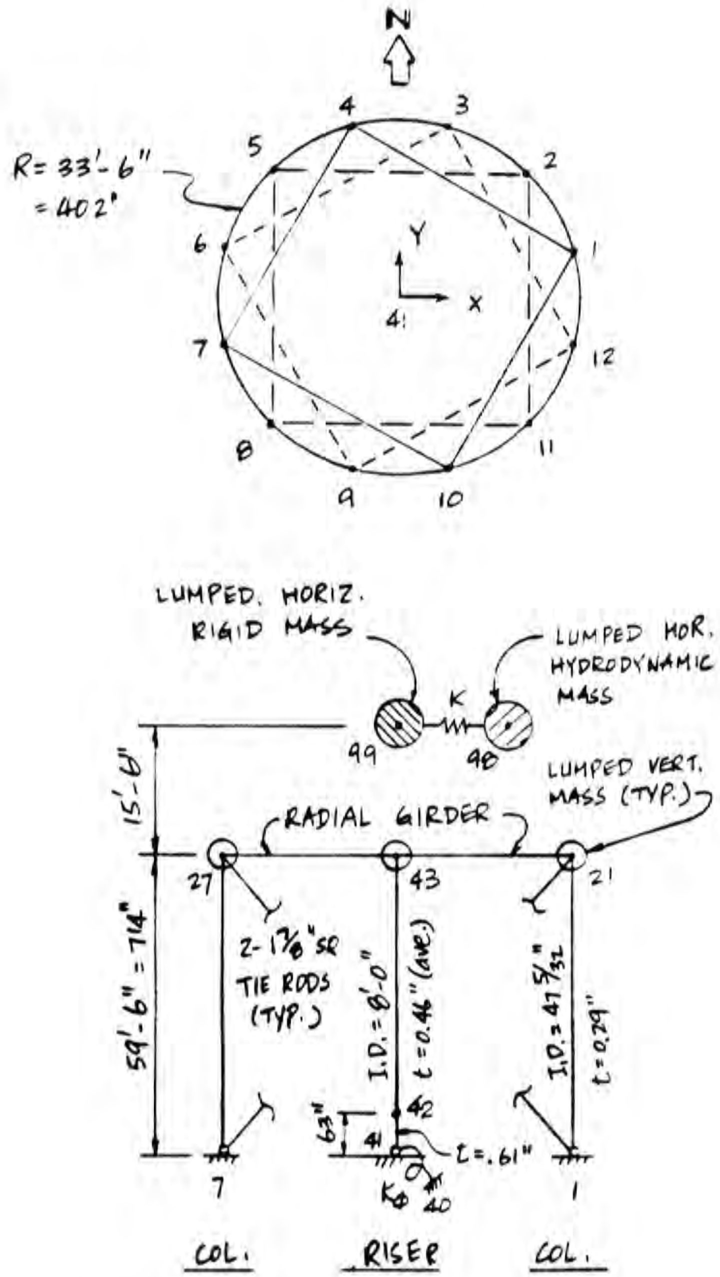
Richmond Highlands #1 Elevated Water Storage Tank
Elevation View

Figure 14.8-1



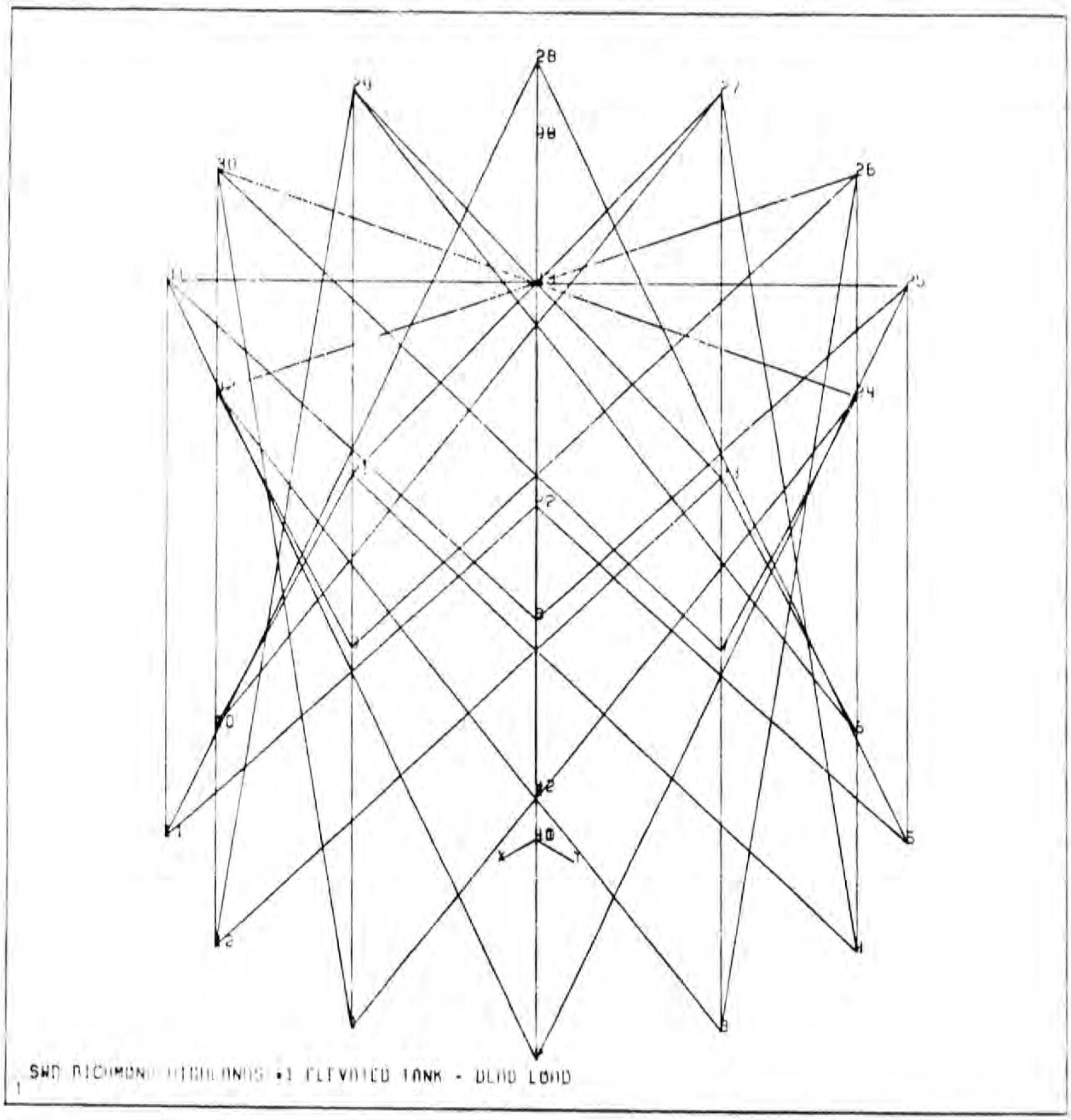
Richmond Highlands #1 Elevated Water Storage Tank
Plan View

Figure 14.8-2



Richmond Highlands #1 Elevated Water Storage Tank Mathematical Model

Figure 14.8-3



Richmond Highlands #1 Elevated Water Storage Tank
Computer Plot

Figure 14.8-4

14.9 Richmond Highlands #2

14.9.1 Facility Description

The Richmond Highlands #2 elevated water storage tank, built in 1958, has a capacity of 2,000,000 gallons. The structure is shown in Figures 14.9-1 to 14.9-4. The elevated tank shell is supported by radial girders resting on sixteen (16) braced columns with a diameter of 42 in. and a cross sectional area of approximately 65 sq. in. The braces have a cross section of 14 sq. in. and are relied upon as compression elements as well as tension elements. There is a central riser with a diameter of 120 in. and a cross section of approximately 308 sq. in. The columns are supported by spread footings with tie beams along the outside perimeter.

The structure has a diameter of approximately 50 ft., and the center of gravity for the structure and contents is located at 73 ft. above grade level.

14.9.2 Analysis

The computer program ANSYS was used in the analysis. The three dimensional model is shown in Figure 14.9-3. The tank shell connection at the top of the columns was modeled by coupling the horizontal radial beams to the top of the columns, and using constraint equations to couple the horizontal displacements without further restraining the rotations. This assumption is justified because at those connections forces are transferred primarily by shear along the tangential surface of the tank. Along this tangential direction the tank shell is very rigid. The model accounts for the ability of the braces to take compression loads.

The connections at the base of each column are modeled as hinges. The riser has eight (8) anchor bolts. Rotational springs were modeled at the base of the riser.

The water content was modeled using Housner's method. The weight of the water is distributed to the columns and riser in accordance to their tributary area.

14.9.3 Analysis Results

The fundamental period of vibration of the tank is 6.5 seconds, and the fluid sloshing period is 5.6 seconds. This places the tank period in the descending branch of the response spectra (Figure 14.2-1). Any softening of the tank (i.e. yielding) will contribute to lowering the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 14.4-1 and 14.4-2. In our opinion, the structure will not remain operable after the Class I earthquake. Yielding and buckling of the braces and yielding of the columns will contribute to instability of the structure. Because in this structure the braces are required to be effective for compression loads, their buckling can contribute to a progressive brittle failure mode of the support structure.

The structure will remain operable after the Class II earthquake. Moderate yielding of the riser anchors will take place resulting in lower effective loads in the structure.

A comparison of design base shears computed using the 1988 UBC Code and the Class I and Class II earthquakes is provided in Table 4.4-3.

14.9.4 Upgrade Recommendations

This structure requires increased lateral strength. It should be noted that if the columns and braces are strengthened beyond a certain limit, problems will arise at the foundation level (soil overstress). A cost-effective strategy is to limit the level of upgrade allowing the braces to undergo some yielding.

These recommended upgrades are:

a) Increase the cross sectional area of the columns by 24 sq. in. each.

b) Increase the cross sectional area of the braces by 4 sq. in. each.

c) Strengthen the connections accordingly, such that the connection capacities exceed the brace capacities.

The estimated cost of upgrading the Richmond Highlands #2 elevated water storage tank, as shown in Table 14.10-2, is $626,300.

Table 14.9-1

Richmond Highlands #2 Elevated Water Storage Tank (High Priority) Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	1.36	Yielding and buckling
Riser Tension	0.84	Adequate
Riser Compression	0.84	Adequate
Riser Base Connection	2.00	Anchors yield
Columns	1.1	Minor yielding
Radial Girder	0.85	Adequate
Soil Under Columns	1.00 (allowable)	Adequate
Soil Under Riser	0.73 (allowable)	Adequate

Elevated Tank Status: **Not Operable**

Estimated Probability of Failure: **70%**

Table 14.9-2

Richmond Highlands #2 Elevated Water Storage Tank (High Priority) Analysis Results

Class II Earthquake

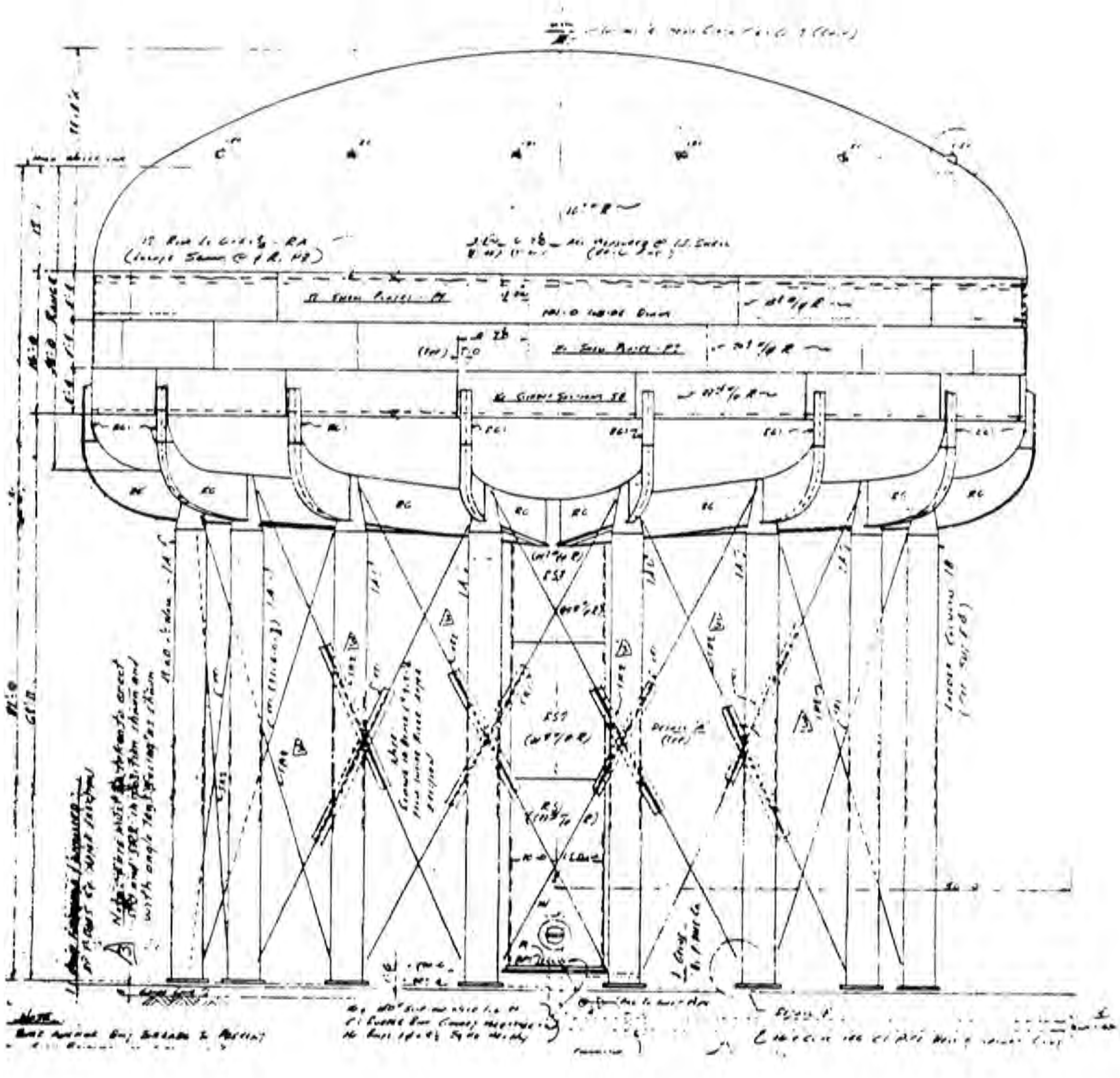
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Diagonal Braces	0.84	Adequate
Riser	<0.84	Adequate
Riser Base Connection	>1.00	Yielding
Columns	<1.00	Adequate
Radial Girder	<0.85	Adequate
Soil Under Columns	<1.00 (allowable)	Adequate
Soil Under Riser	<0.73 (allowable)	Adequate

Elevated Tank Status: **Operable**

Table 14.9-3

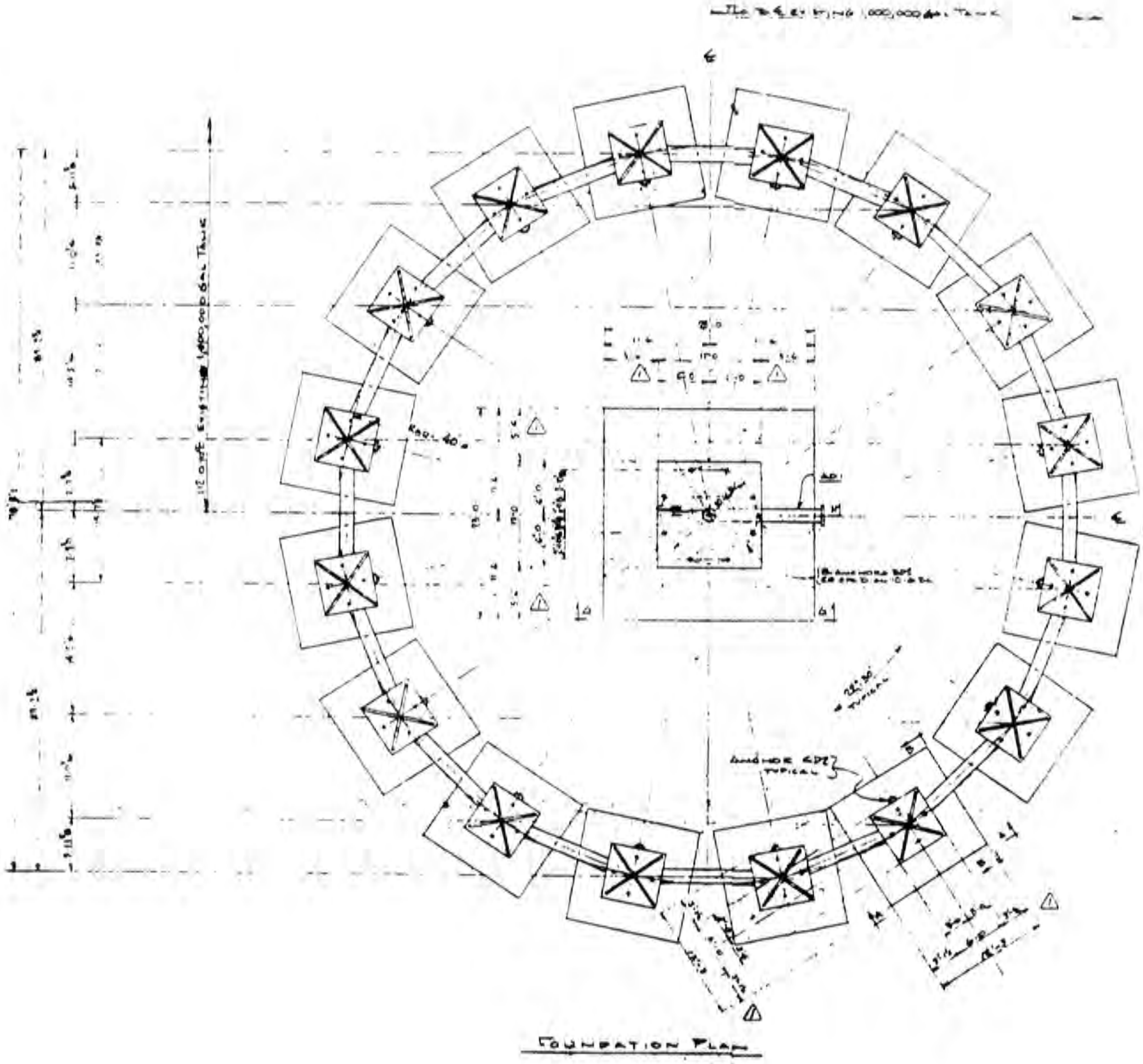
Richmond Highlands #2 Elevated Water Storage Tank
Design Base Shear Comparison

1988 UBC Code	1,743 kips
Class I Earthquake	694 kips
Class II Earthquake	430 kips



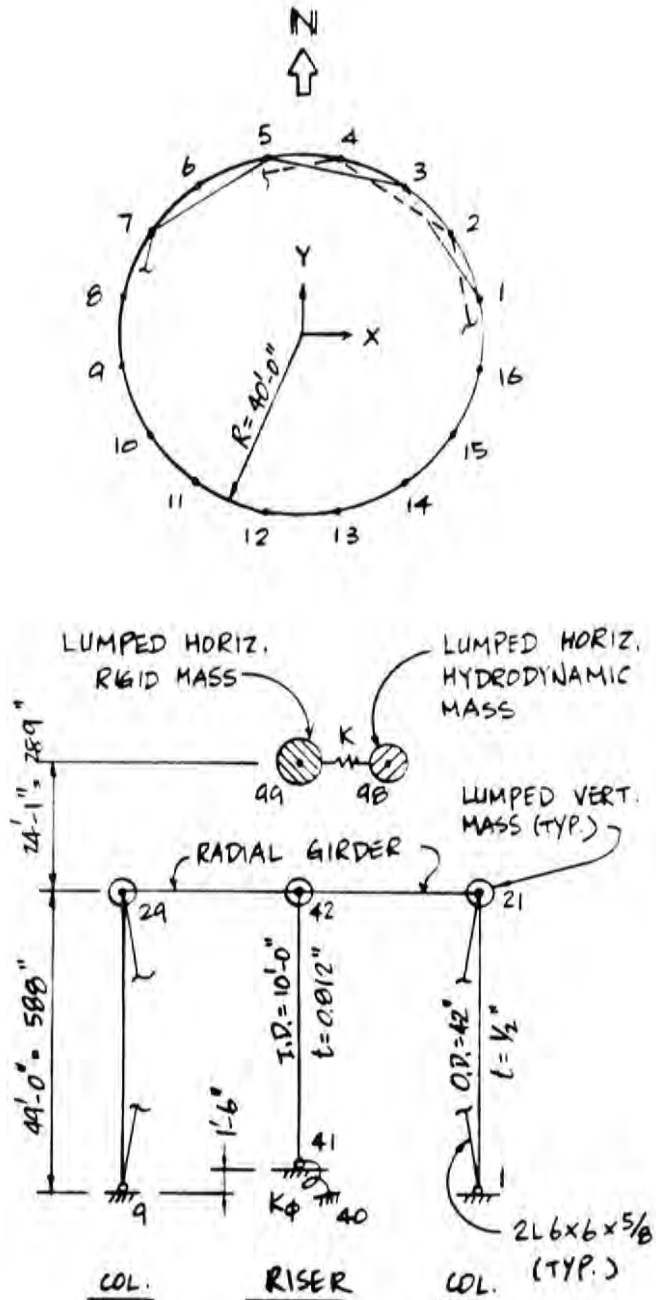
Richmond Highlands #2 Elevated Water Storage Tank
Elevation View

Figure 14.9-1



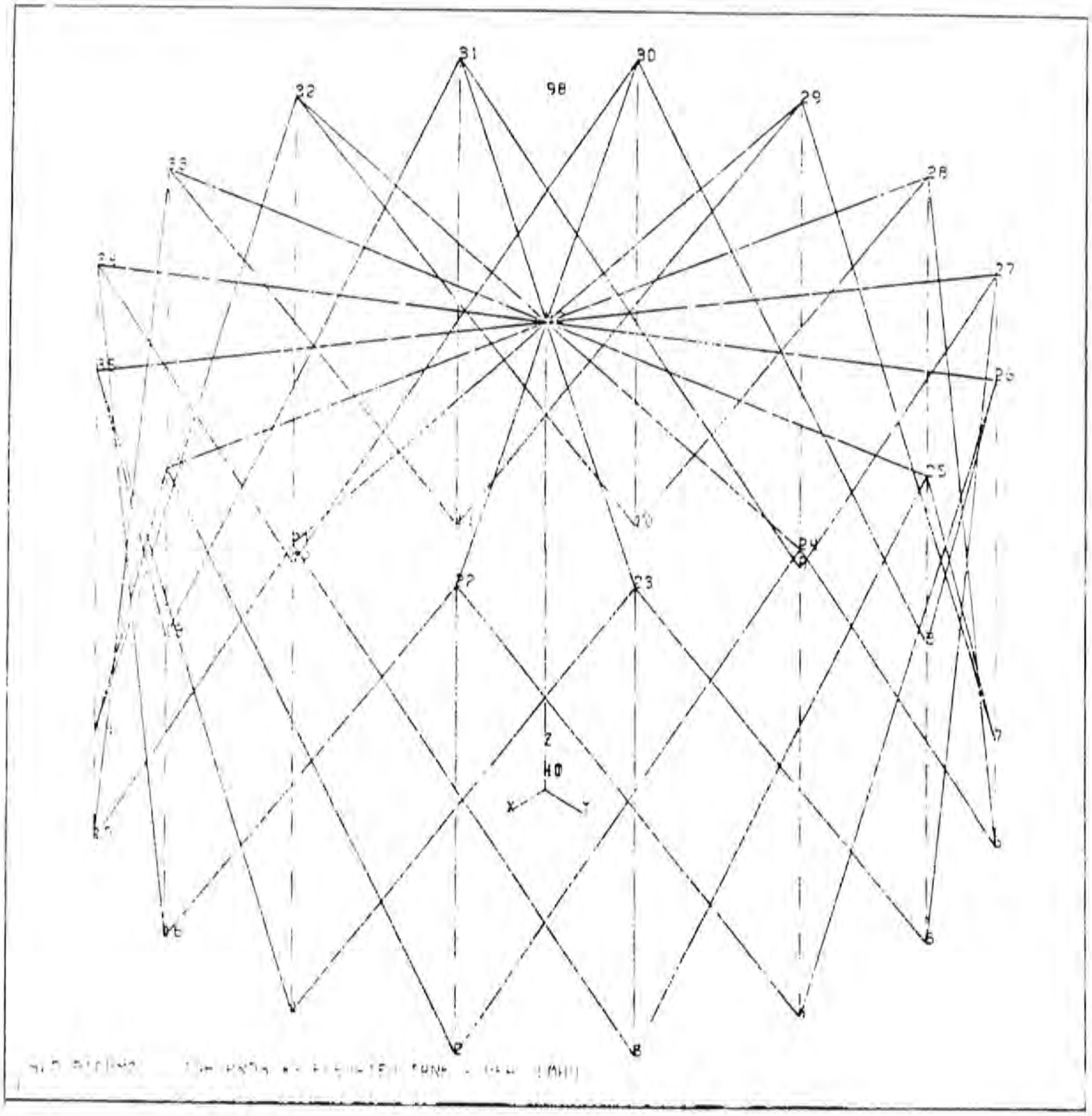
Richmond Highlands #2 Elevated Water Storage Tank
Plan View

Figure 14.9-2



Richmond Highlands #2 Elevated Water Storage Tank
Mathematical Model

Figure 14.9-3



Richmond Highlands #2 Elevated Water Storage Tank
Computer Plot

Figure 14.9-4

14.10 Summary and Conclusions

14.10.1 Geotechnical

The elevated tanks appear to be generally founded upon native, glacially consolidated sediments which are not susceptible to the development of liquefaction or earthquake-induced landslides. Tanks at Magnolia Bluff and S.W. Myrtle may be possibly founded upon shallow fill soils. As such, there may be some potential for differential settlement beneath the foundations of these structures in a Class I earthquake. However, the absence of major signs of distress to the foundations of these structures or adjacent facilities would tend to suggest that any underlying fill is relatively competent and that there does not appear to be a high hazard of earthquake-induced differential settlement beneath these tank foundations.

It is recommended that any future rehabilitation of these tanks include a site-specific geotechnical evaluation, conducted with either borings or test pits, to assess the amount and competency of any fill that may be present at the sites.

14.10.2 Structural

The Seattle Water Department's elevated water tanks have been evaluated for seismic vulnerability when subjected to Class I and Class II earthquakes.

The evaluation was hampered by missing information. No original design calculations were available for review, structural drawings were incomplete, and material test data was limited or non-existent. These inconveniences are to be expected when reviewing structures 30 to 70 years old such as these.

In order to proceed with the vulnerability assessments, many assumptions had to be made at every step. While because of our experience, we feel the assumptions made were reasonable and justifiable, conclusions reached should be interpreted as our professional opinion.

As shown in Table 14.10-1, out of the seven elevated tanks reviewed, all will undergo significant damage if subjected to the Class I earthquake. While the amount of damage expected is hard to ascertain, SW Myrtle Nos. 1 and 2, Maple Leaf, and Richmond Highlands No. 1 are the most vulnerable.

When the tanks are subjected to the Class II earthquake, most of them are expected to undergo some damage but remain operable. SW Myrtle Nos. 1 and 2, and Richmond Highlands No. 1 are not expected to remain operable following the Class II earthquake.

Table 14.10-2 presents a summary of cost estimates developed for seismic upgrade of the tanks. Included in the engineering costs, in addition to the design engineering costs, are those costs associated with architectural considerations, planning and review board meetings, direct costs, and dealing with anticipated interferences such as private property boundaries, underground utilities, and other anticipated physical constraints encountered in upgrade projects such as these.

Table 14.10-1

**Summary Operability Assessment
Elevated Water Storage Tanks**

Elevated Tank	Class I Earthquake	Class II Earthquake
Beverly Park	Operable	Operable
Magnolia Bluff	Not Operable	Operable
Maple Leaf	Not Operable	Operable
SW Myrtle #1	Not Operable	Not Operable
SW Myrtle #2	Not Operable	Not Operable
Richmond Highlands #1	Not Operable	Not Operable
Richmond Highlands #2	Not Operable	Operable

Table 14.10-2

Summary Cost Estimate
Elevated Water Storage Tanks
(Based on 1989 Dollars)

	Estimated Costs						
Elevated Tank	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	Accuracy of Estimate
Beverly Park	$ 62,000	$ 49,000	$ 180,000	$ 291,000	$ 14,600	$ 305,600	±30%
Magnolia Bluff	62,000	49,000	360,000	471,000	29,200	500,200	±30%
Maple Leaf	73,000	55,000	278,000	406,000	22,500	428,500	±30%
SW Myrtle #1	60,000	49,000	300,000	409,000	24,300	433,300	±30%
SW Myrtle #2	88,000	65,000	400,000	553,000	32,400	585,400	±30%
Richmond Highlands #1	73,000	55,000	336,000	464,000	27,200	491,200	±30%
Richmond Highlands #2	81,000	61,000	448,000	590,000	36,300	626,300	±30%
Total Cost	$499,000	$383,000	$2,302,000	$3,184,000	$186,500	$3,370,500	

15.0 STANDPIPES

15.1 Facility Overview

The City of Seattle Water Department (SWD) has nine (9) water storage standpipes ranging in capacity from 318,000 to 1,4000,000 gallons (Table 15.1-1). The standpipes are located throughout the urban area on seven high ground locations.

Because of their size and location, these standpipes must maintain their integrity during and following a major earthquake. Their failure could result in catastrophic flooding and potential loss of life.

The standpipes are a fundamental component of the water system lifeline. As such, they must remain operable after a major earthquake to provide fire fighting capabilities, maintain sanitary conditions and to provide drinking water.

The standpipes were constructed between 1901 and 1933. They are all made of steel, and most of them are anchored.

Table 15.1-1

Seattle Water Department
Water Storage Standpipes

Name	Location	Date of Construction	Overflow Elevation	Capacity in Gallons at Overflow
SW Barton	38th Ave. S.W. & S.W. Barton St.	1927	316	1,400,000
SW Charlestown	39th Ave. S.W. & S.W. Charlestown St.	1927	488	1,000,000
Foy	N. 145th St. & Dayton Ave. N.	1933	580	1,000,000
Queen Anne #1	Warren Ave. N. & Lee St.	1901	520	318,000
Queen Anne #2	Warren Ave. N. & Lee St.	1904	520	883,000
SW Trenton - North	4th Ave. S.W. & S.W. Trenton St.	1932	320	1,193,000
SW Trenton - South	4th Ave. S.W. & S.W. Trenton St.	1932	320	1,193,000
Volunteer Park	14th Ave. E. and E. Prospect St.	1907	520	883,000
Woodland Park	Phinney Ave. N. near N. 53rd St.	1925	420	1,000,000
		Total Capacity :		8,870,000

15.2 Geologic Hazard Assessment

All of the standpipes are located in Zone II as identified on the seismic hazard map in Figure 4.3-1. Subsurface geologic conditions in Zone II typically consist of glacially consolidated sediments. These materials are typically not susceptible to liquefaction during the design earthquakes. Additionally, the slight to moderate topographic relief at most standpipe locations would generally preclude the development of earthquake-induced landslides.

Three of the standpipes, S.W. Barton, S.W. Charleston, and Volunteer Park, may be partially founded upon fill soils. The geological site reconnaissance indicated that the upper 2 to 3 feet of the fill appeared to be loose. Borings were not available at these locations to confirm the competency of the foundation bearing stratum. Accordingly, if these facilities are located upon fill, there is a potential for the development of differential settlement of the foundations. It is anticipated that differential settlements under the Class I earthquake should be on the order of 0.5 in. or less. This assumes a fill depth of less than 20 ft. and a relative density of 45 percent within the fill. Both of these assumptions are quite conservative. However, any rehabilitation of these standpipes should include an evaluation of the foundation conditions.

The S.W. Trenton standpipe is located adjacent to an oversteepened fill slope. There is potential that this fill slope could experience shallow surficial sloughing during either a Class I or Class II earthquake. The development of surficial slumping on this slope would most likely affect the security fence surrounding the facility. Any surficial slumping, however, is not anticipated to affect the overall stability of the standpipe foundation.

In conclusion, all of the standpipes appear to be located in areas where there is relatively minimal risk of the development of liquefaction or landsliding beneath the foundations of the structures. Shallow surficial landsliding may occur outside of the S.W. Trenton standpipe; however, such movement is not anticipated to endanger the foundation of the facility. The standpipes located at S.W. Barton, S.W. Charlestown, and Volunteer Park may be partially founded upon fill soils. Accordingly, these foundations may potentially experience differential settlement under a Class I earthquake.

15.3 Barton

15.3.1 Facility Description

The Barton water storage standpipe, constructed in 1927, has a capacity of 1,400,000 gallons. The structure is shown in Figure 15.3-1. The tank has a diameter of approximately 80 ft. and a height of 38 ft. The cylindrical shell ranges in thickness from 1/4 in. to 3/4 in. The bottom plate is 5/16-in. thick. The tank is supported on a mat foundation, 3 to 4-ft. thick and 55-ft. in diameter. The tank is not anchored.

15.3.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15.3-2. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring.

15.3.3 Analysis Results

The fundamental period of vibration of the standpipe is 0.05 seconds, and the fluid sloshing period is 5.26 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 15.3-1 and 15.3-2. In our opinion the structure will not remain operable after the Class I earthquake. High loads and lack of anchorage will contribute to softening of the structure and will compromise its overall stability.

The structure will remain operable after the Class II earthquake.

Table 15.3-3 shows a comparison of design shears using different criteria.

15.3.4 Upgrade Recommendations

The recommended upgrade solution for this standpipe is to introduce enough anchor bolts to provide a 4.9 kips per foot capacity around the perimeter of the tank. To do so would result in an increased overturning moment that the current foundation is capable of withstanding. In addition, the tank would now be stable against uplift and the shell stress ratio would be less than one.

The estimated cost for upgrading the Barton standpipe, as shown in Table 15.11-2, is $117,900.

Table 15.3-1

**Barton Water Storage Standpipe (High Priority)
Analysis Results**

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell (comprehensive buckling)	1.11	Not adequate (tank unstable - could uplift)
Anchor Bolts	N.A.	No anchor bolts
Soil	0.50 (allowable)	Adequate

Standpipe Status: **Not Operable**

Estimated Probability of Failure: **60%**

Table 15.3-2

Barton Water Storage Standpipe (High Priority)
Analysis Results

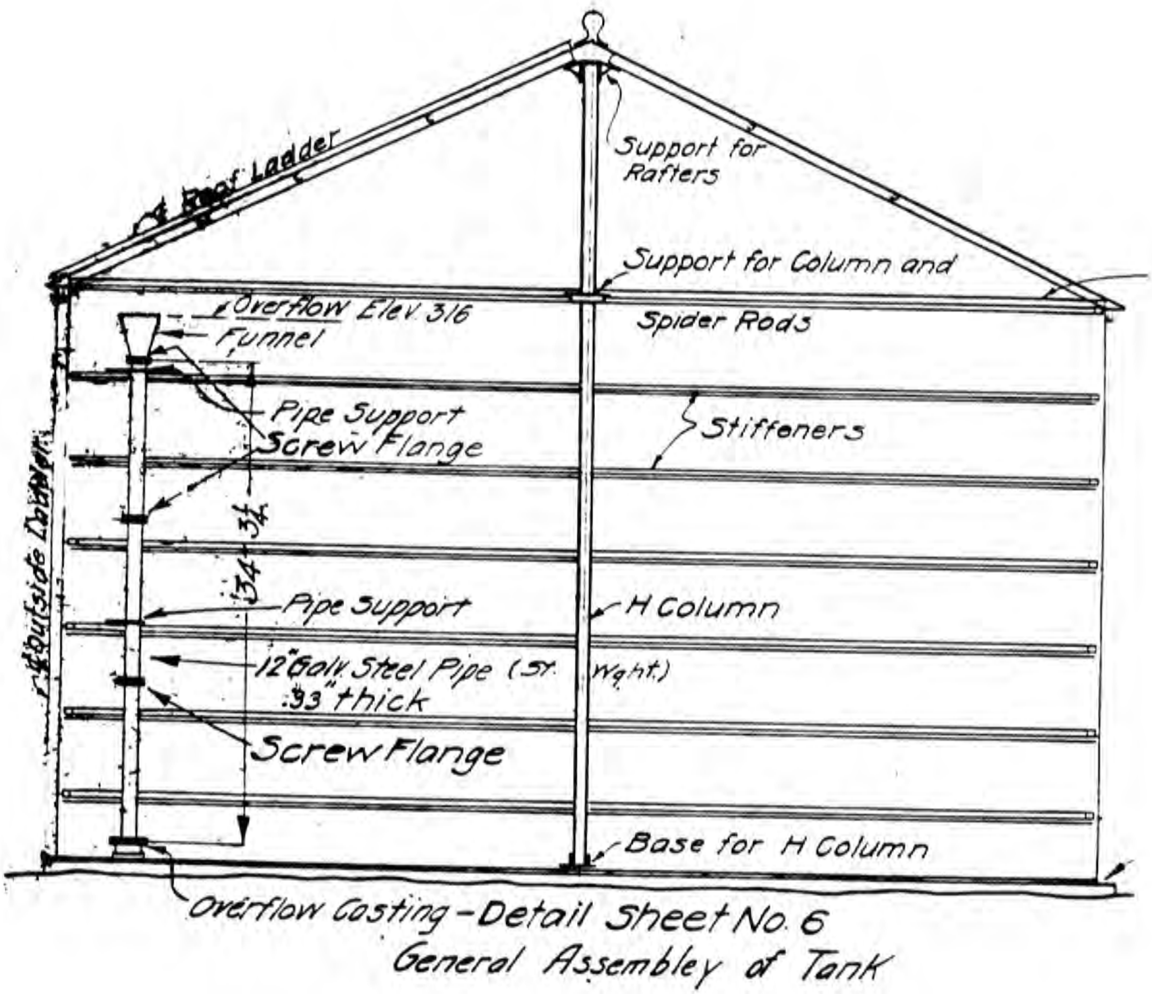
Class II Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.68	Adequate
Anchor Bolts	N.A.	No anchor bolts
Soil	<1.0 (allowable)	Adequate
Standpipe Status	**Operable**	

Table 15.3-3

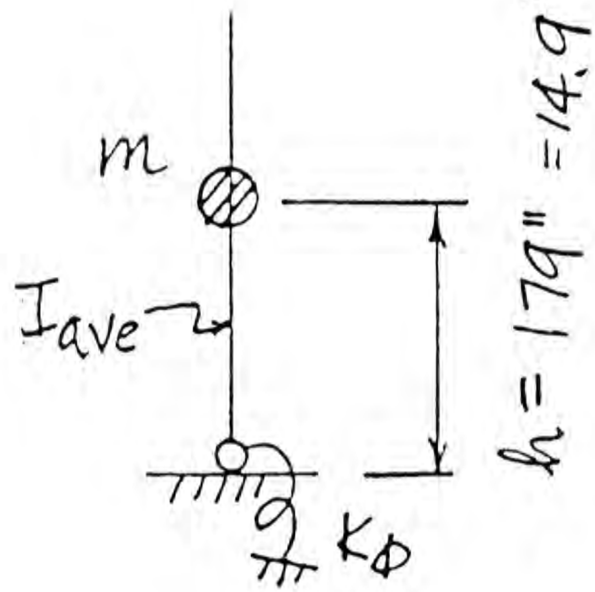
Barton Water Storage Standpipe (High Priority)
Design Base Shear Comparison

1988 UBC Code	3,167 kips
Class I Earthquake	2,923 kips
Class II Earthquake	1,812 kips



Barton Water Storage Standpipe

Figure 15.3-1



$K_\phi = 8.95E9$ "K/rad
$m = 17.0$ K-sec²/in

Barton Water Storage Standpipe
Mathematical Model

Figure 15.3-2

15.4 Charlestown

15.4.1 Facility Description

The Charlestown water storage standpipe, constructed in 1927, has a capacity of 883,000 gallons. The structure is shown in Figure 15.4-1. The structure has a diameter of approximately 50 ft. and a height of 62 ft. The cylindrical shell ranges in thickness from 1/4 in. to 13/16 in. The bottom plate is 5/16 in. thick. The tank is supported on a mat foundation, assumed to be 4 ft. thick and 56 ft. in diameter. Anchorage is provided by twenty 2 in. diameter bolts with 3 ft. 6 in. embedment.

15.4.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15.4-2. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring.

15.4.3 Analysis Results

The fundamental period of vibration of the standpipe is 0.16 seconds, and the fluid sloshing period is 4 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 15.4-1 and 15.4-2. In our opinion, the structure will not remain operable after either the Class I or the Class II earthquakes. Yielding of the anchor bolts and rotation of the footing will contribute to softening of the structure and will compromise its overall stability.

Table 15.4-3 shows a comparison of design shears using different criteria.

15.4.4 Upgrade Recommendations

The recommended upgrade solution for this standpipe is not simply to increase the number of anchor bolts. To do so would result in an increased overturning moment that the current foundation could not withstand. The upgrade solution is complicated by interference with the property line and neighboring structures. A possible fix would be to install twenty (20) new diagonal TS 6 x 6 x 3/8-in. braces which tie into a circumferential ring girder. These braces can take both tension and compression. They are equally distributed around the circumference of the tank and are anchored by a new ring foundation footing as shown in Figure 15.4-3.

The estimated cost to upgrade the Charlestown standpipe, as shown in Table 15.11-2, is $384,200.

Table 15.4-1

Charlestown Water Storage Standpipe (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.49	Adequate
Anchor	3.50	Failure
Soil	0.92 (allowable)	Adequate

Standpipe Status: **Not Operable**

Estimated Probability of Failure: **70%**

Table 15.4-2

Charlestown Water Storage Standpipe (High Priority)
Analysis Results

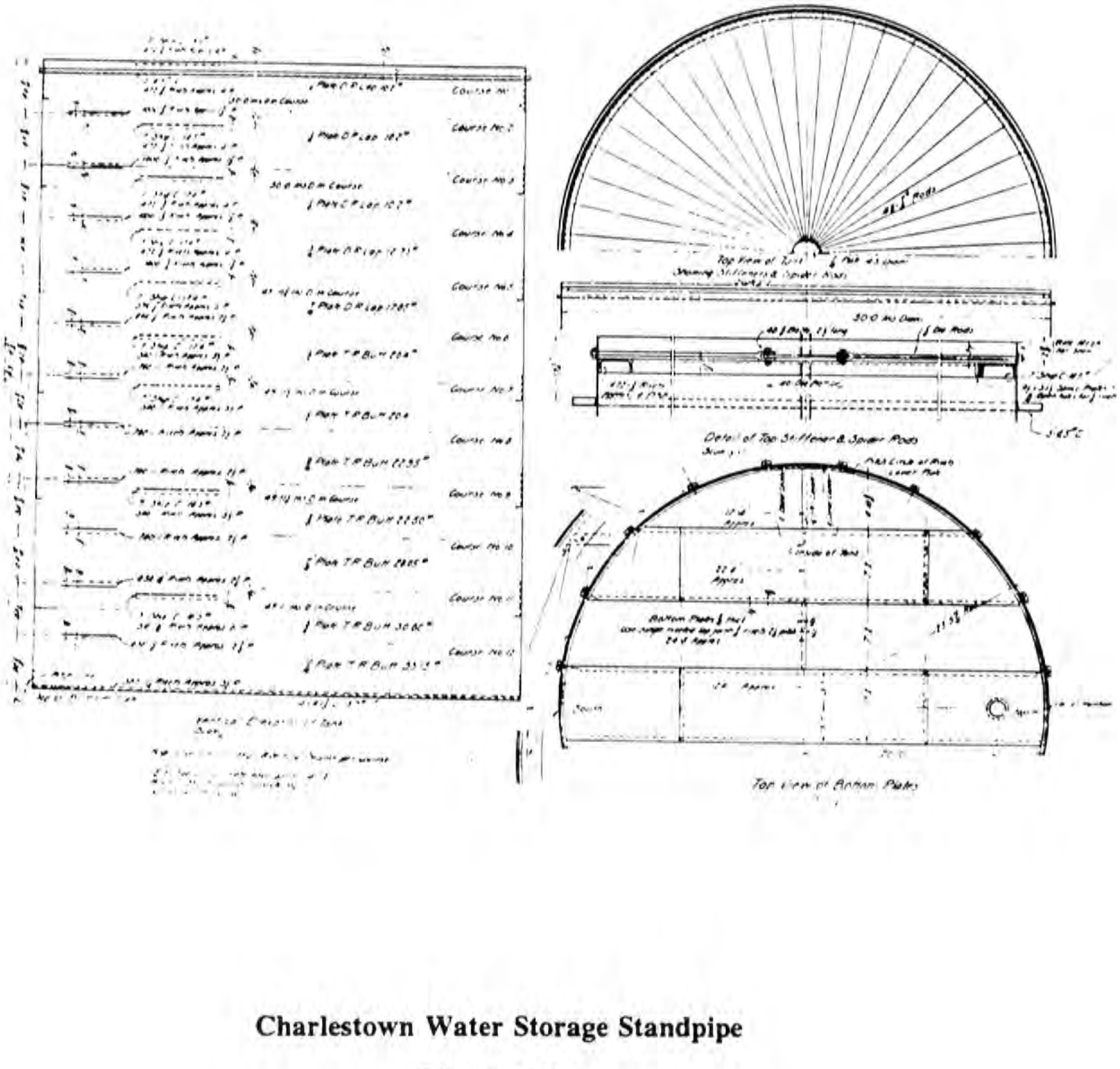
Class II Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	<0.4	Adequate
Anchor Bolts	2.05	Failure
Soil	<0.7 (allowable)	Adequate
Standpipe Status	**Not Operable**	

Table 15.4-3

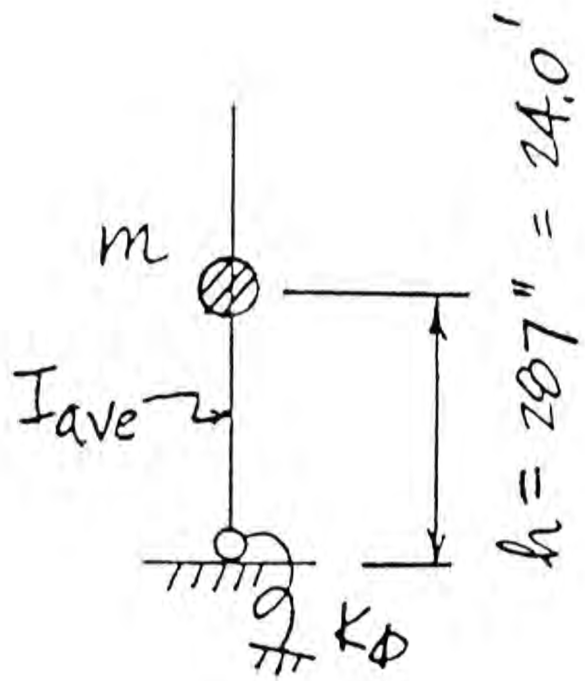
Charlestown Water Storage Standpipe (High Priority)
Design Base Shear Comparison

1988 UBC Code	2,038 kips
Class I Earthquake	4,180 kips
Class II Earthquake	2,592 kips



Charlestown Water Storage Standpipe

Figure 15.4-1

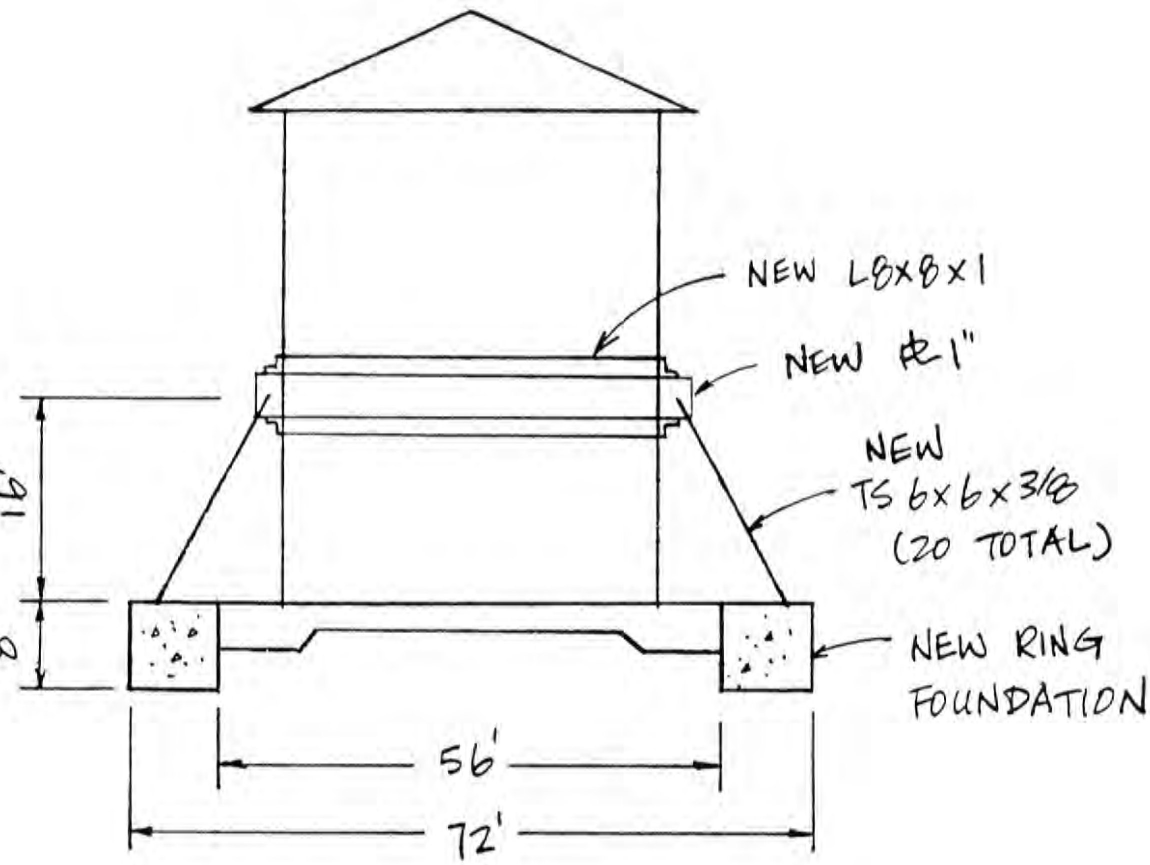


$$K_\phi = 2.56E9 \ ''k/rad$$

$$m = 17.9 \ k\text{-}sec^2/in$$

Charlestown Water Storage Standpipe
Mathematical Model

Figure 15.4-2



Charlestown Water Storage Standpipe
Conceptual Upgrade

Figure 15.4-3

15.5 Foy

15.5.1 Facility Description

The Foy water storage standpipe, constructed in 1933, has a capacity of 1,000,000 gallons. The structure is shown in Figure 15.5-1. The standpipe has a diameter of approximately 46 ft. and a height of 83 ft. The cylindrical shell ranges in thickness from 5/16 in. to 1 in. The bottom plate is 5/16-in. thick. The tank is supported on a mat foundation, 3- to 4-ft. thick and 52-ft. in diameter. Anchorage is provided by twenty (20) 2-1/2 in. diameter bolts with 3 ft. 6 in. embedment.

15.5.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15.5-2. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring.

15.5.3 Analysis Results

The fundamental period of vibration of the standpipe is 0.26 seconds, and the fluid sloshing period is 1.1 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 15.5-1 and 15.5-2. In our opinion, the structure will not remain operable after the Class I and the Class II earthquakes. Yielding of the anchor bolts and rotation of the footing will contribute to softening of the structure and will compromise its overall stability.

Table 15.5-3 shows a comparison of design shears using different criteria.

15.5.4 Upgrade Recommendations

The recommended solution for this standpipe is not simply to increase the number of anchor bolts. To do so would result in an increased overturning moment that the current foundation could not withstand. Two possible fixes are:

a) Lower the water level to at least El. 464.5 feet,

or

b) Install twenty (20) new TS 6 x 6 x 3/8-in. braces which tie into a circumferential ring girder. These braces can take both tension and compression. They are equally distributed around the circumference of the tank and are anchored by a new ring foundation footing as shown in Figure 15.5-3.

The estimated cost to upgrade the Foy standpipe, as shown in Table 15.11-2, is $384,200.

Table 15.5-1

Foy Water Storage Standpipe (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.76	Adequate
Anchor Bolts	4.40	Yield or rupture
Soil	1.17 (allowable)	Minor overstress

Standpipe Status: **Not Operable**

Estimated Probability of Failure: **80%**

Table 15.5-2

Foy Water Storage Standpipe (High Priority)
Analysis Results

Class II Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	<0.5	Adequate
Anchor Bolts	2.62	Yielding
Soil	<0.9 (allowable)	Adequate
Standpipe Status	**Not Operable**	

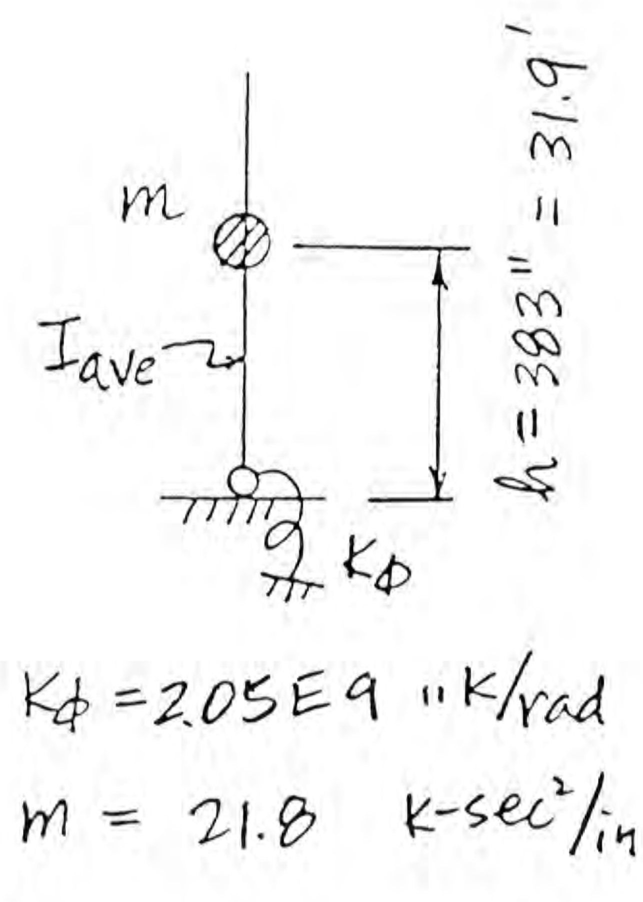
Table 15.5-4

Foy Water Storage Standpipe (High Priority)
Design Base Shear Comparison

1988 UBC Code	2,328 kips
Class I Earthquake	6,215 kips
Class II Earthquake	3,853 kips

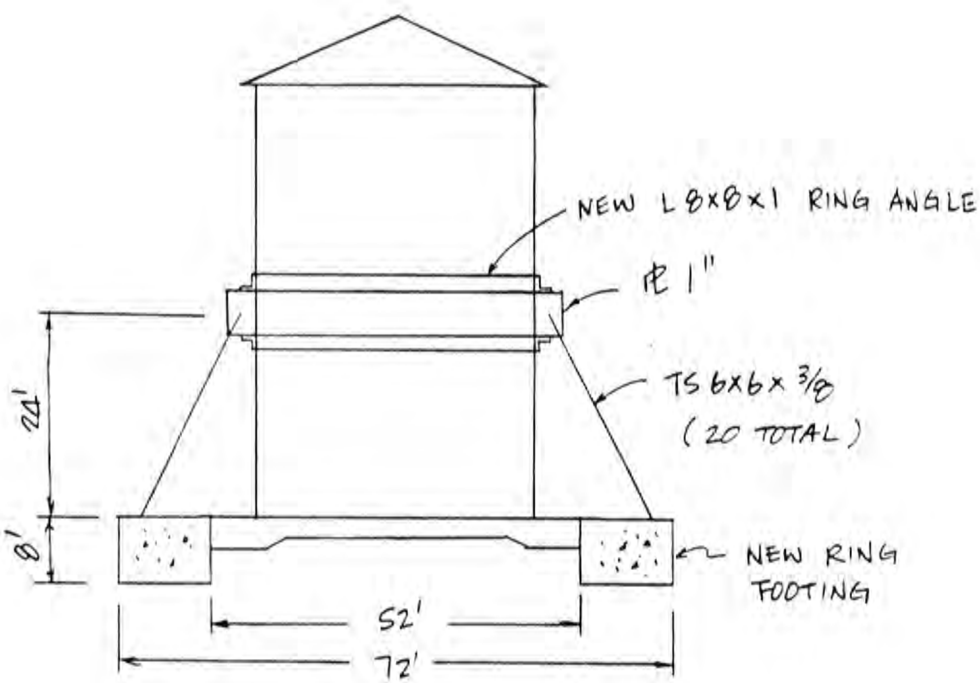
Foy Water Storage Standpipe

Figure 15.5-1



Foy Water Storage Standpipe
Mathematical Model

Figure 15.5-2



Foy Water Storage Standpipe
Conceptual Upgrade

Figure 15.5-3

15.6 Queen Anne #1

15.6.1 Facility Description

The Queen Anne #1 water storage standpipe, constructed in 1901, has a capacity of 318,000 gallons. The structure is shown in Figure 15.6-1. The standpipe has a diameter of approximately 30 ft. and a height of 64 ft. The cylindrical shell ranges in thickness from 1/4 in. to 5/8 in. The bottom plate is 1/4 in. thick. The tank is supported on a mat foundation, 1-1/2 ft. thick and 42 ft. in diameter. Anchorage is provided by eight (8) 1-1/4 in. diameter bolts with 3 ft. 6 in. embedment.

The standpipe is enclosed by a cylindrical reinforced concrete structure with a 12-in. thick wall and a diameter of 18 ft. with minimal reinforcement. The concrete structure and the standpipe share the same foundation.

15.6.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15.6-2. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring.

The concrete structure was evaluated in accordance with ACI 318-83.

15.6.3 Analysis Results

The fundamental period of vibration of the combined structure is 0.32 seconds. This places the structural period in the ascending branch of the response spectra. Any softening of the structure (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 15.6-1 and 15.6-2. In our opinion, the structure will not remain operable after either the Class I or the Class II earthquakes. Failure of the anchor bolts, concrete cracking, and rotation of the footing will contribute to softening the structure and will compromise its overall stability. It should also be noted that the structure shows signs of steel corrosion and concrete spalling.

Table 15.6-3 shows a comparison of design shears using different criteria.

15.6.4 Upgrade Recommendations

The recommended upgrade solution for this standpipe is not simple. The task is complicated because of the very large overstress found in different components, and the space limitations resulting from the proximity to Queen Anne #2 and the property boundary.

A possible fix would be to install nine (9) new TS 8 x 8 x ½-in. braces which tie into a circumferential ring girder. These braces can take both tension and compression. They are equally distributed around the circumference of the tank and are anchored by a new footing common to Queen Anne #1 and Queen Anne #2 as shown in Figure 15.6-3.

The estimated upgrade cost for Queen Anne #1 is coupled to the cost of the recommended upgrade for Queen Anne #2. The estimated total cost for upgrading these two standpipes, as shown in Table 15.11-2, is $753,500.

Table 15.6-1

Queen Anne #1 Water Storage Standpipe (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.24	Adequate
Anchor Bolts	4.70	Anchor failure
Soil	5.20 (allowable)	Major overstress
Concrete Building Shear	2.10	Shear failure

Standpipe Status: **Not Operable**

Estimated Probability of Failure: **85%**

Table 15.6-2

Queen Anne #1 Water Storage Standpipe (High Priority)
Analysis Results

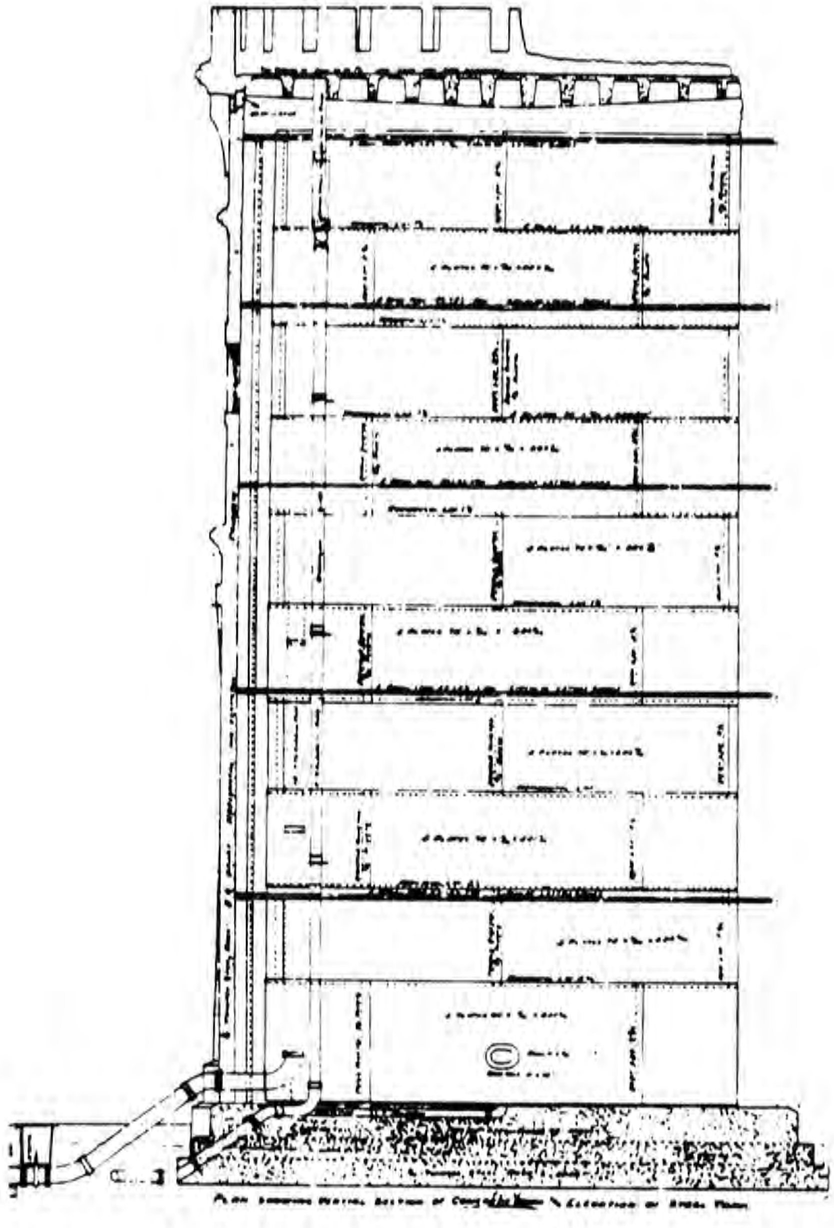
Class II Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell		Adequate
Anchor Bolts	1.75	Failure
Soil	1.31 (allowable)	Overstress
Concrete Structure	1.10	Overstress
Standpipe Status	**Not Operable**	

Table 15.6-3

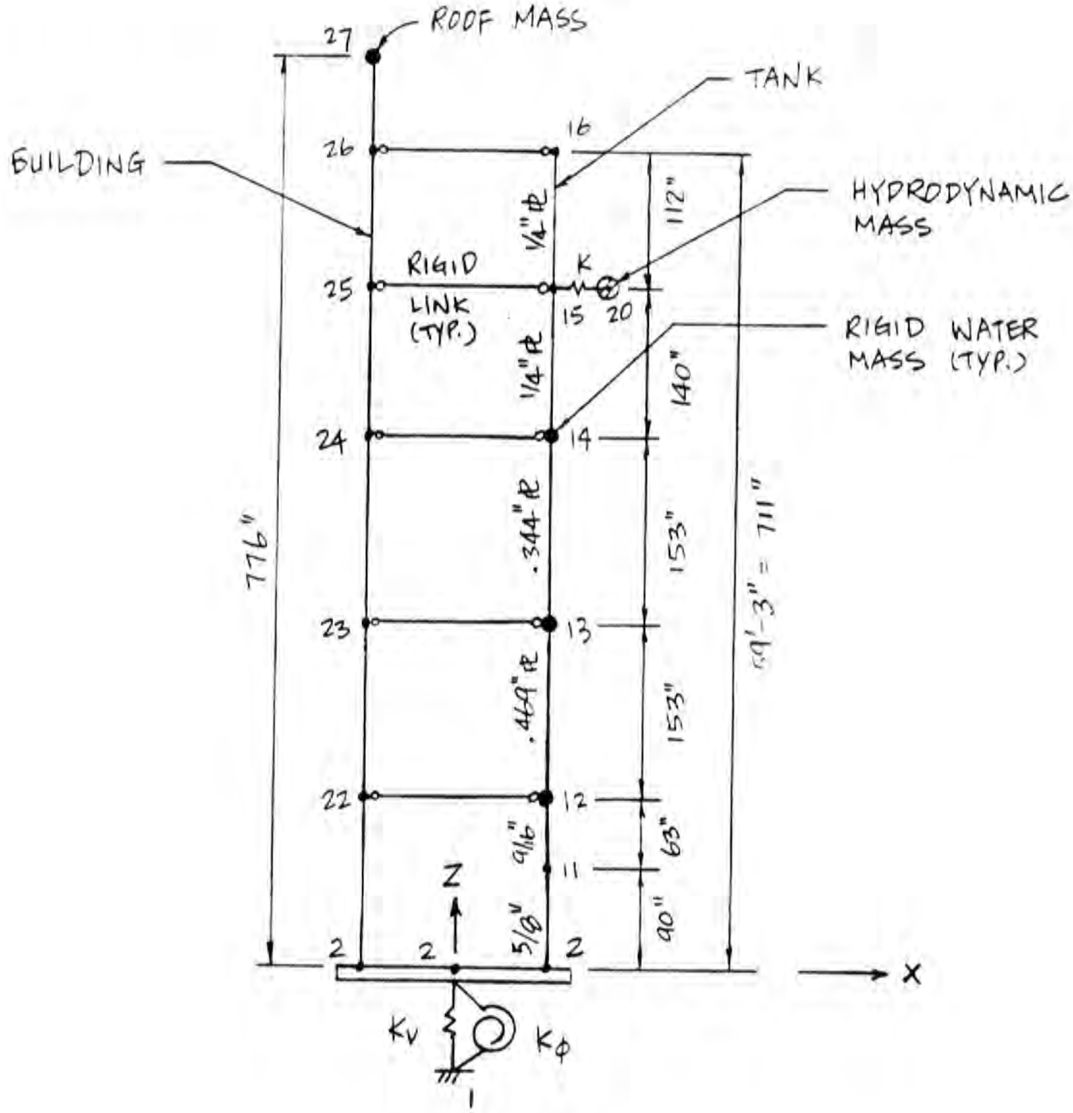
**Queen Anne #1 Water Storage Standpipe (High Priority)
Design Base Shear Comparison**

1988 UBC Code	2,252 kips
Class I Earthquake	2,098 kips
Class II Earthquake	1,297 kips



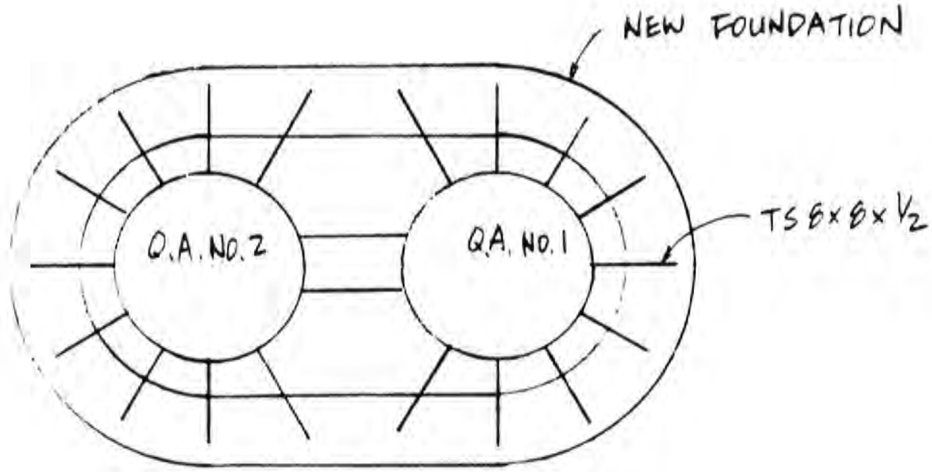
Queen Anne #1 Water Storage Standpipe

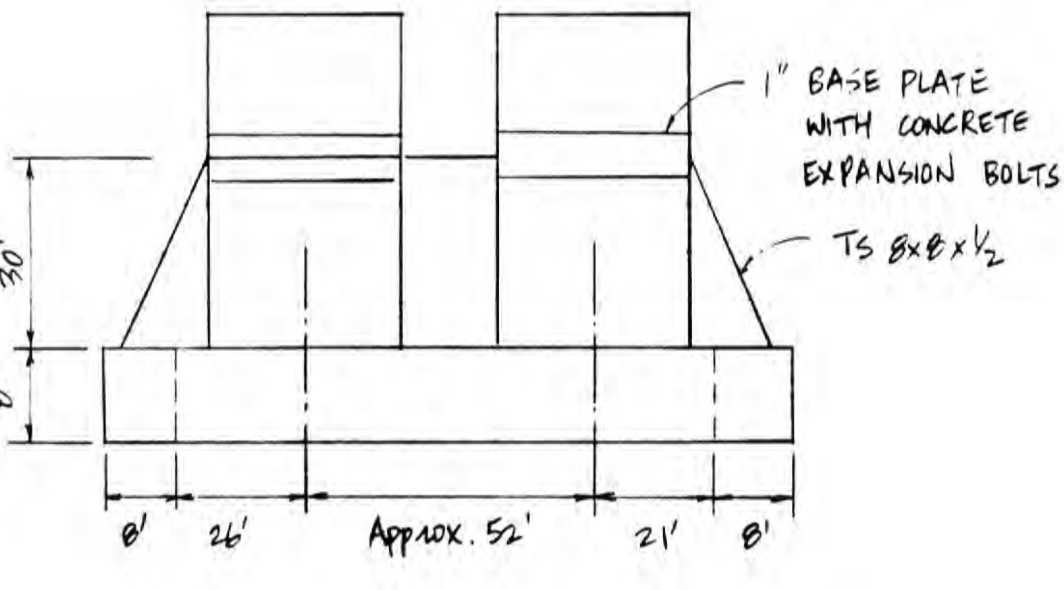
Figure 15.6-1



Queen Anne #1 Water Storage Standpipe
Mathematical Model

Figure 15.6-2





Queen Anne #1 and Queen Anne #2 Water Storage Standpipe Conceptual Upgrade

Figure 15.6-3

15.7 Queen Anne #2

15.7.1 Facility Description

The Queen Anne #2 water storage standpipe, constructed in 1932, has a capacity of 883,000 gallons. The structure, shown in Figure 15.7-1, has a diameter of approximately 50 ft. and a height of 65 ft. The cylindrical shell ranges in thickness from 5/16 in. to 15/16 in. The bottom plate is 1/2-in. thick. The tank is supported on a mat foundation, assumed to be 4-ft. thick and 52-ft. in diameter. Anchorage is provided by twenty (20) 1-1/4 in. diameter bolts with 3 ft. 6 in. embedment.

15.7.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15.7-2. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring.

15.7.3 Analysis Results

The fundamental period of vibration of the standpipe is 0.16 seconds, and the fluid sloshing period is 4.16 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 15.7-1 and 15.7-2. In our opinion the structure will not remain operable after either the Class I or the Class II earthquakes. Yielding of the anchor bolts and rotation of the footing will contribute to softening of the structure and will compromise its overall stability.

Table 15.7-3 shows a comparison of design shears using different criteria.

15.7.4 Upgrade Recommendations

The recommended solution for this standpipe is not simply to increase the number of anchor bolts. To do so would result in an increased overturning moment that the current foundation could not withstand. The upgrade solution is complicated by interference with the property line and neighboring structures. It is recommended that nine (9) new TS 8 x 8 x ½-in. braces be installed which tie into a circumferential ring girder. These braces can take both tension and compression. They are equally distributed around the circumference of the tanks and are anchored by a new footing as shown in Figure 15.6-3.

The estimated upgrade cost for Queen Anne #2 is coupled to the cost of the recommended upgrade for Queen Anne #1. The estimated total cost for upgrading these two standpipes, as shown in Table 15.11-2, is $735,500.

Table 15.7-1

Queen Anne #2 Water Storage Standpipe (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.43	Adequate
Anchor Bolts	7.90	Failure
Soil	1.23 (allowable)	Minor overstress

Standpipe Status: **Not Operable**

Estimated Probability of Failure: **75%**

Table 15.7-2

Queen Anne #2 Water Storage Standpipe (High Priority)
Analysis Results

Class II Earthquake

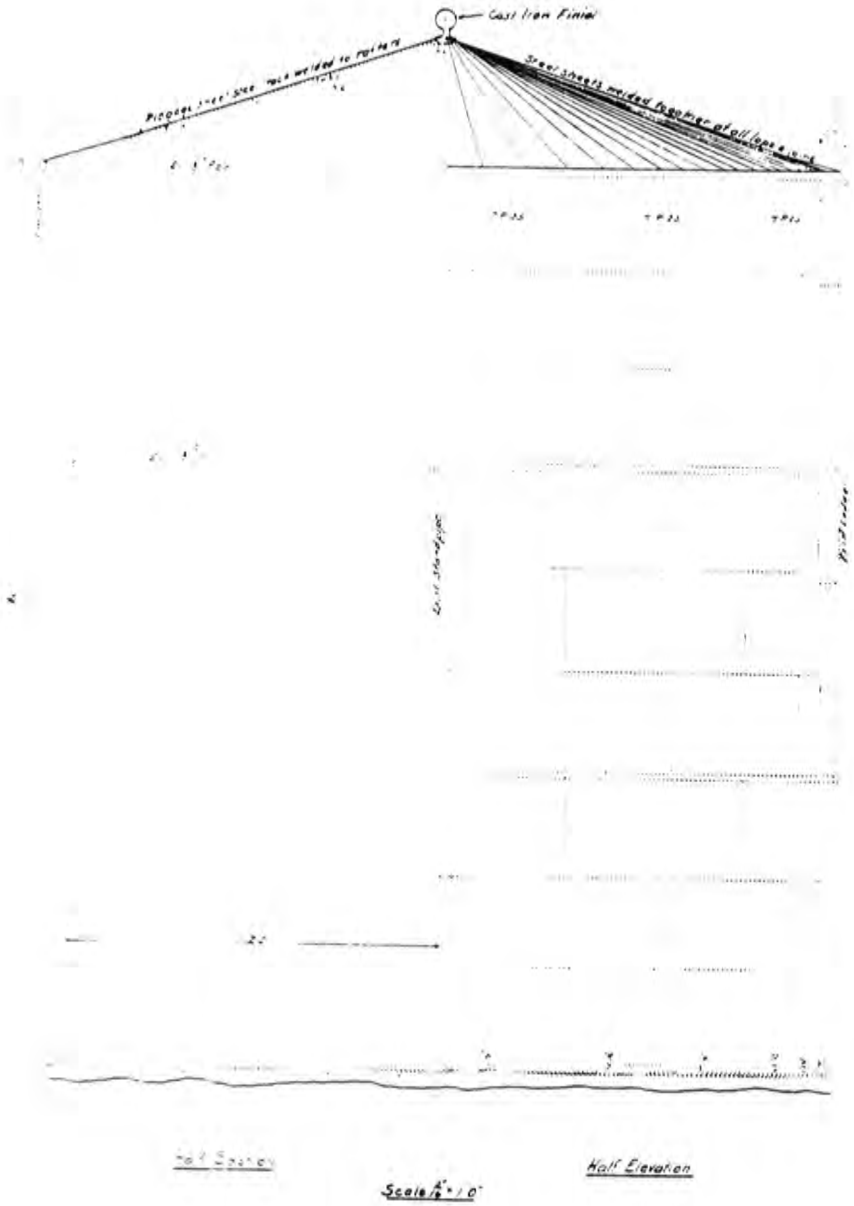
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	<0.25	Adequate
Anchor Bolts	4.46	Failure
Soil	<0.9 (allowable)	Adequate

Standpipe Status: **Not Operable**

Table 15.7-3

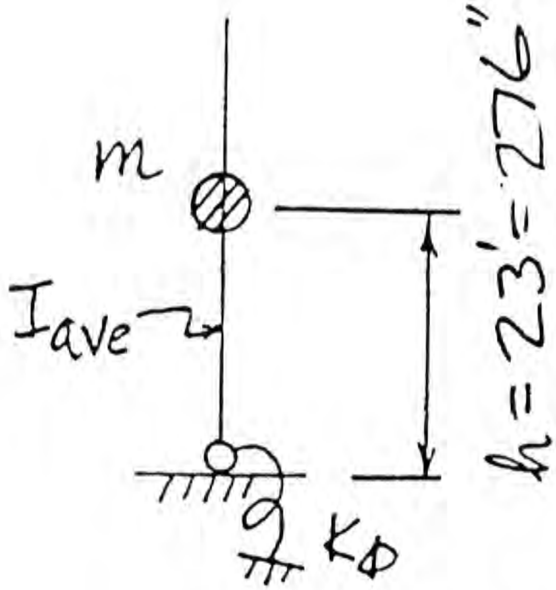
**Queen Anne #2 Water Storage Standpipe (High Priority)
Design Base Shear Comparison**

1988 UBC Code	1,974 kips
Class I Earthquake	4,001 kips
Class II Earthquake	2,481 kips



Queen Anne #2 Water Storage Standpipe

Figure 15.7-1



$$K_\phi = 2.05\text{E}9 \text{ "K/rad}$$

$$m = 17.1 \text{ k-sec}^2/\text{in}$$

Queen Anne #2 Water Storage Standpipe
Mathematical Model

Figure 15.7-2

15.8 S.W. Trenton North and South

15.8.1 Facility Description

The SW Trenton North and South water storage standpipes have a capacity of 1,193,000 gallons. The structure, shown in Figure 15.8-1, has a diameter of approximately 92 ft. and a height of 36 ft. The cylindrical shell ranges in thickness from 5/16 in. to 11/16 in. The bottom plate is 5/16-in. thick. The tank is supported on a mat foundation, assumed to be 4-ft. thick and 97-ft. in diameter. No anchorage is provided.

15.8.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15.8-2. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring.

15.8.3 Analysis Results

The fundamental period of vibration of the standpipe is 0.02 seconds, and the fluid sloshing period is 6.25 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Table 15.8-1. In our opinion, the structures will remain operable after the Class 1 earthquake.

Table 15.8-3 shows a comparison of design shears using different criteria.

15.8.4 Upgrade Recommendations

No upgrade is required for these standpipes. Furthermore, the water level can be increased by 3 ft. above the current overflow level to a total height of 27 ft. Because water in these standpipes can back up more than 2 ft. above the overflow, it should be emphasized that the maximum water level must be limited to 27 ft., particularly for situations where overflow conditions persist for extended periods of time.

Table 15.8-1

S.W. Trenton North and South Water Storage Standpipes (High Priority) Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.07	Adequate
Anchor Bolts	N/A	Not anchored
Soil	0.31	Adequate

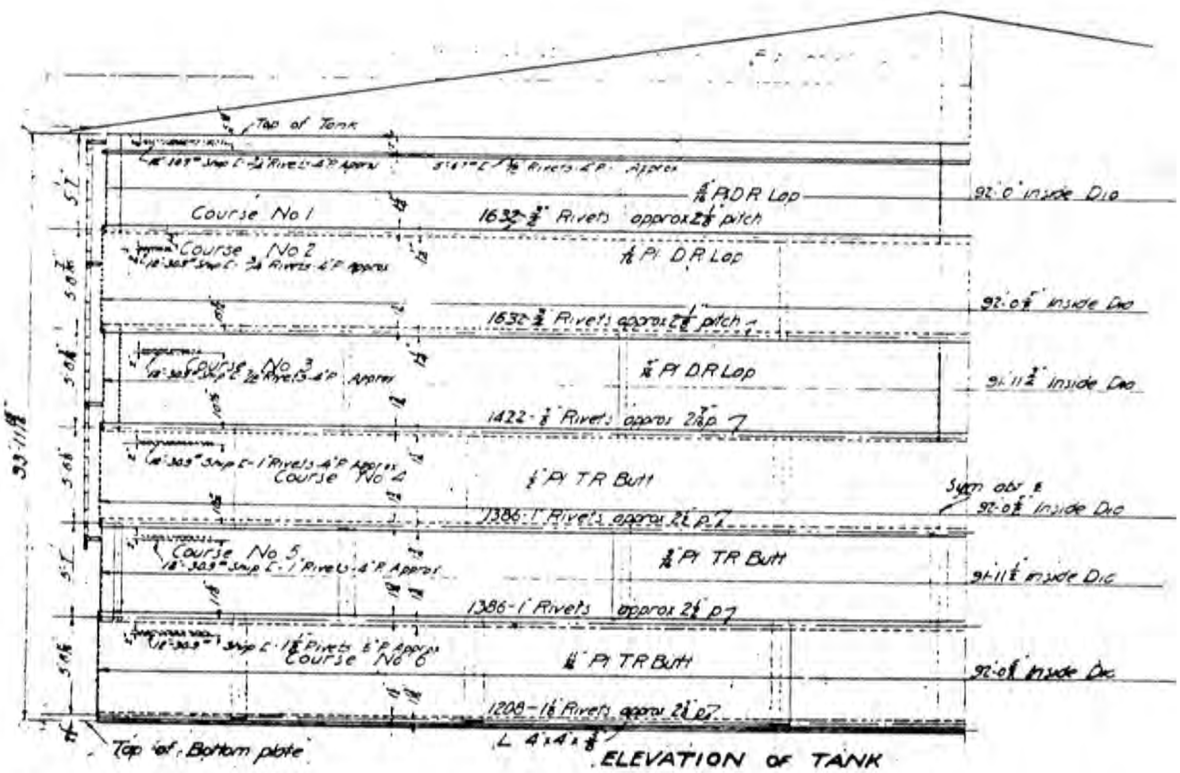
Standpipe Status: **Operable**

Estimated Probability of Failure: **10%**

Table 15.8-2

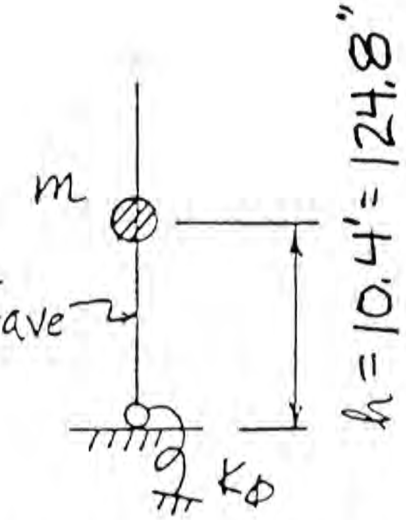
S.W. Trenton North and South Water Storage Standpipes (High Priority) Design Base Shear Comparison

1988 UBC Code	2,760 kips
Class I Earthquake	1,280 kips
Class II Earthquake	794 kips



SW Trenton North and South Water Storage Standpipes

Figure 15.8-1



$K_\phi = 1.33E10 \text{ ''K/rad}$

$m = 9.0 \text{ K-sec}^2/\text{in}$

SW Trenton North and South Water Storage Standpipes
Mathematical Model

Figure 15.8-2

15.9 Volunteer Park

15.9.1 Facility Description

The Volunteer Park water storage standpipe, constructed in 1907, has a capacity of 883,000 gallons. The structure, shown in Figure 15.9-1, has a diameter of approximately 50 ft. and a height of 66 ft. The cylindrical shell ranges in thickness from 5/16 in. to 15/16 in. The bottom plate is 1/2-in. thick. The tank is supported on a mat foundation, 2-1/2-ft. thick and 70-ft. in diameter. Anchorage is provided by eight (8) 1-1/4 in. diameter bolts.

The standpipe is enclosed by a cylindrical brick masonry structure with 16 to 24-in. thick walls and a diameter of 62 ft. with minimal reinforcement. The masonry structure and the standpipe share the same foundation. The standpipe and the masonry structure are braced together at regular intervals.

15.9.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15-9. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as a hydrodynamic mass with an equivalent spring.

The masonry structure was evaluated in accordance with UBC-1988.

15.9.3 Analysis Results

The fundamental period of vibration of the combined structure is 0.27 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the structure (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 15.9-1 and 15.9-2. In our opinion, the structure will not remain operable after either the Class I or the Class II earthquakes. Failure of the anchor bolts, masonry shear cracking, and rotation of the footing will contribute to softening of the structure and will compromise its overall stability.

Table 15.9-3 shows a comparison of design shears using different criteria.

15.9.4 Upgrade Recommendations

The upgrade of this standpipe is very costly. The task is complicated because of the very large overstress found in different components, and the space limitations between the masonry building and the standpipe shell. Because the Volunteer Park standpipe is a historical structure, however, it is desirable to develop a remedial upgrade solution.

One potential upgrade scheme involves removing the roof of the structure for access, and introducing a new support structure in the space between the masonry building and the standpipe shell. A new foundation will have to be provided for the new support structure. The building will have to be attached to the new structure in a cosmetically-acceptable manner, and the roof will have to be replaced (see Figure 15.9-3).

The estimate cost of ugrading the Volunteer Park standpipe and its enclosure structure, as shown in Table 15.11-2, is $2,317,000.

Table 15.9-1

Volunteer Park Water Storage Standpipe (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.32	Adequate
Anchor Bolts	7.4	Anchor failure
Soil	2.6 (allowable)	Major overstress
Masonry Building Shear	4.7	Shear failure

Standpipe Status: **Not Operable**

Estimated Probability of Failure: **90%**

Table 15.9-2

Volunteer Park Water Storage Standpipe (High Priority)
Analysis Results

Class II Earthquake

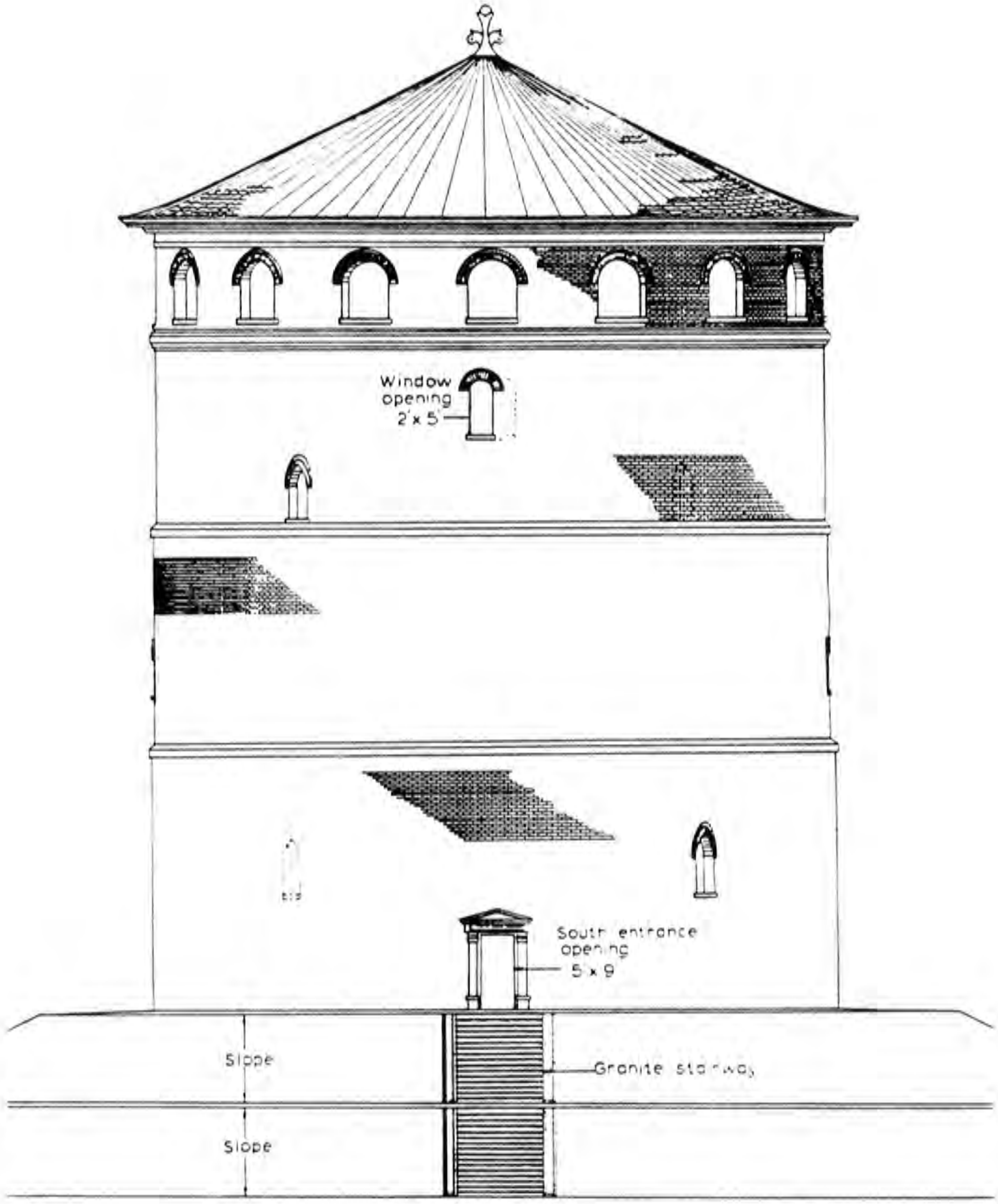
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	<0.2	Adequate
Anchor Bolts	3.89	Failure
Soil	0.75 (allowable)	Adequate
Masonry Building Shear	1.35	Overstress

Standpipe Status: **Not Operable**

Table 15.9-3

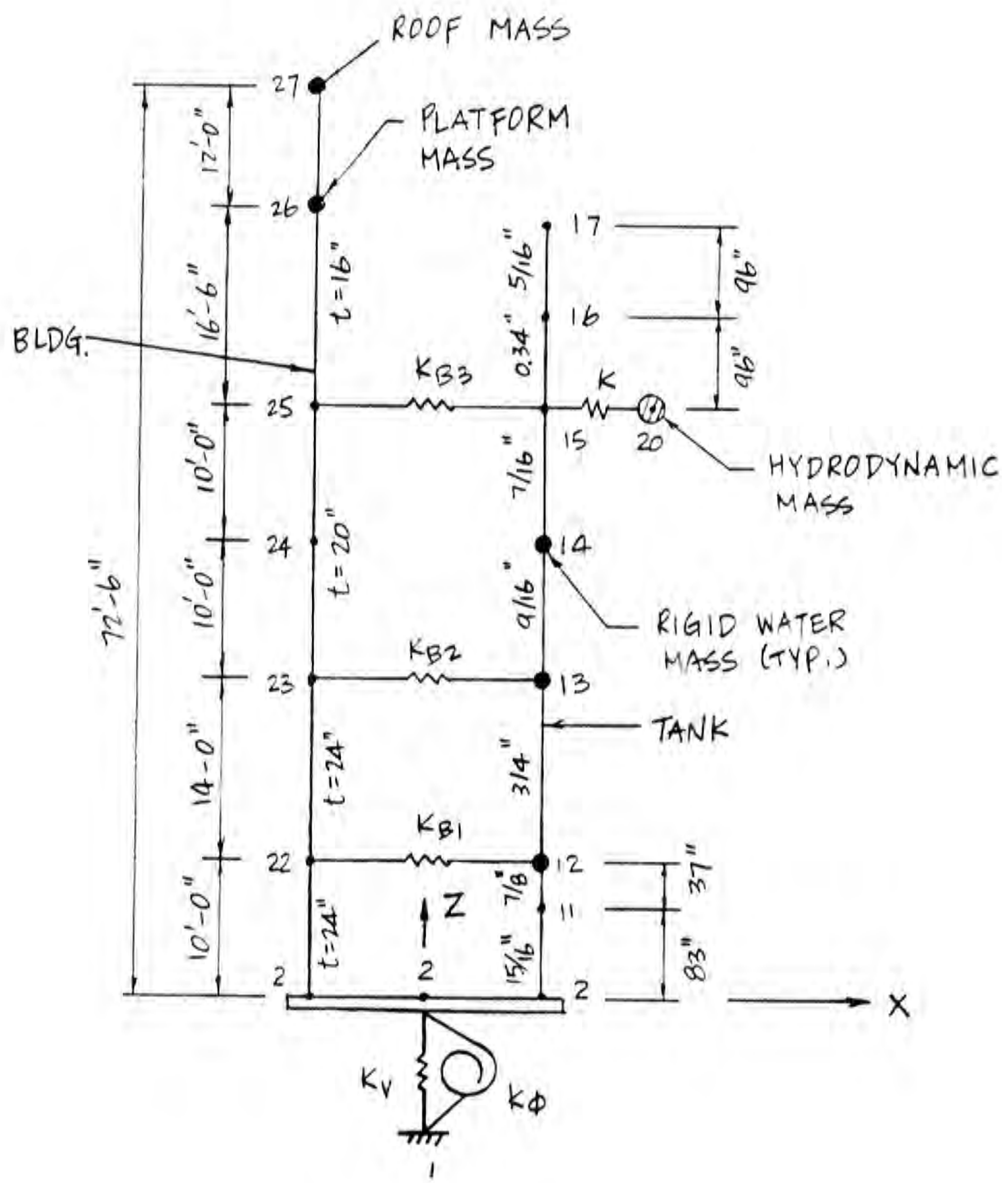
Volunteer Park Water Storage Standpipe (High Priority)
Design Base Shear Comparison

1988 UBC Code	2,252 kips
Class I Earthquake	2,098 kips
Class II Earthquake	1,297 kips



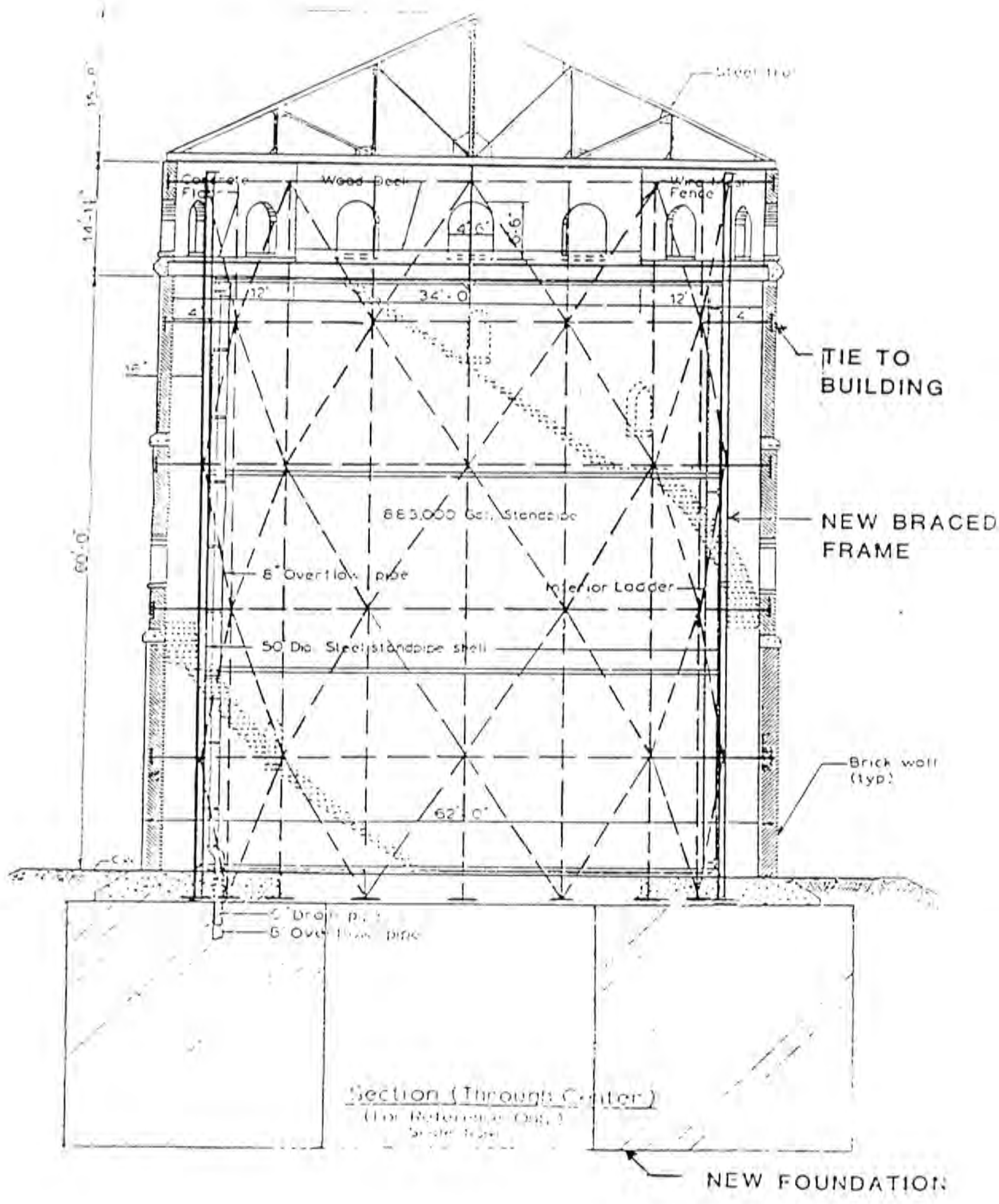
Volunteer Park Water Storage Standpipe

Figure 15.9-1



**Volunteer Park Water Storage Standpipe
Mathematical Model**

Figure 15.9-2



Volunteer Park Water Storage Standpipe
Conceptual Upgrade

Figure 15.9-3

15.10 Woodland Park

15.10.1 Facility Description

The Woodland Park water storage standpipe, constructed in 1925, has a capacity of 1,000,000 gallons. The structure, shown in Figure 15.10-1, has a diameter of approximately 50 ft. and a height of 75 ft. The cylindrical shell ranges in thickness from 1/4 in. to 3/4 in. The bottom plate is 5/16-in. thick. The tank is supported on a mat foundation, assumed to be 2-ft. thick and 52-ft. in diameter. Anchorage is provided by twenty (20) 1-1/2-in. diameter bolts with 3 ft. 6 in. embedment.

15.10.2 Analysis and Procedure

The computer program ANSYS was used for the analysis. The simplified model is shown in Figure 15.10-2. It should be noted that standpipes are "short period" structures and thus, any modeling assumptions which underestimate their rigidity are non-conservative because they can result in artificially lower spectral accelerations. The effect of soil-structure interaction cannot be ignored. The flexibility of the soil will increase the period of the standpipe and introduce a larger spectral acceleration.

The water content was modeled using Housner's method. Most of the water moves rigidly with the tank, and the sloshing water is modeled as an hydrodynamic mass with an equivalent spring.

15.10.3 Analysis Results

The fundamental period of vibration of the standpipe is 0.17 seconds, and the fluid sloshing period is 4 seconds. This places the tank period in the ascending branch of the response spectra. Any softening of the tank (i.e. yielding) will contribute to increasing the effective seismic loads in the structure.

Stress ratios for critical elements are presented in Tables 15.10-1 and 15.10-2. In our opinion the structure will not remain operable after either the Class I or the Class II earthquakes. Yielding or rupture of the anchor bolts will contribute to softening of the structure and will compromise its overall stability.

Table 15.10-3 shows a comparison of design shears using different criteria.

15.10.4 Upgrade Recommendations

The recommended solution for this standpipe is not simply to increase the number of anchor bolts. To do so would result in an increased overturning moment that the current foundation could not withstand. The upgrade solution is complicated by interference with the property line and neighboring structures. A possible design fix would be to install twenty (20) new diagonal TS 6 x 6 x 3/8-in. braces which tie into a circumferential ring girder. These braces can take both tension and compression. They are equally distributed around the circumference of the tank and are anchored by a new ring foundation footing, similar to that shown in Figure 15.4-3.

The estimated cost of upgrading the Woodland Park standpipe, as shown in Table 15.11-2, is $384,200.

Table 15.10-1

Woodland Park Water Storage Standpipe (High Priority)
Analysis Results

Class I Earthquake

Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	0.47	Adequate
Anchor Bolts	5.40	Failure
Soil	0.57 (allowable)	Overstress

Standpipe Status: **Not Operable**

Estimated Probability of Failure: **85%**

Table 15.10-2

**Woodland Park Water Storage Standpipe (High Priority)
Analysis Results**

Class II Earthquake

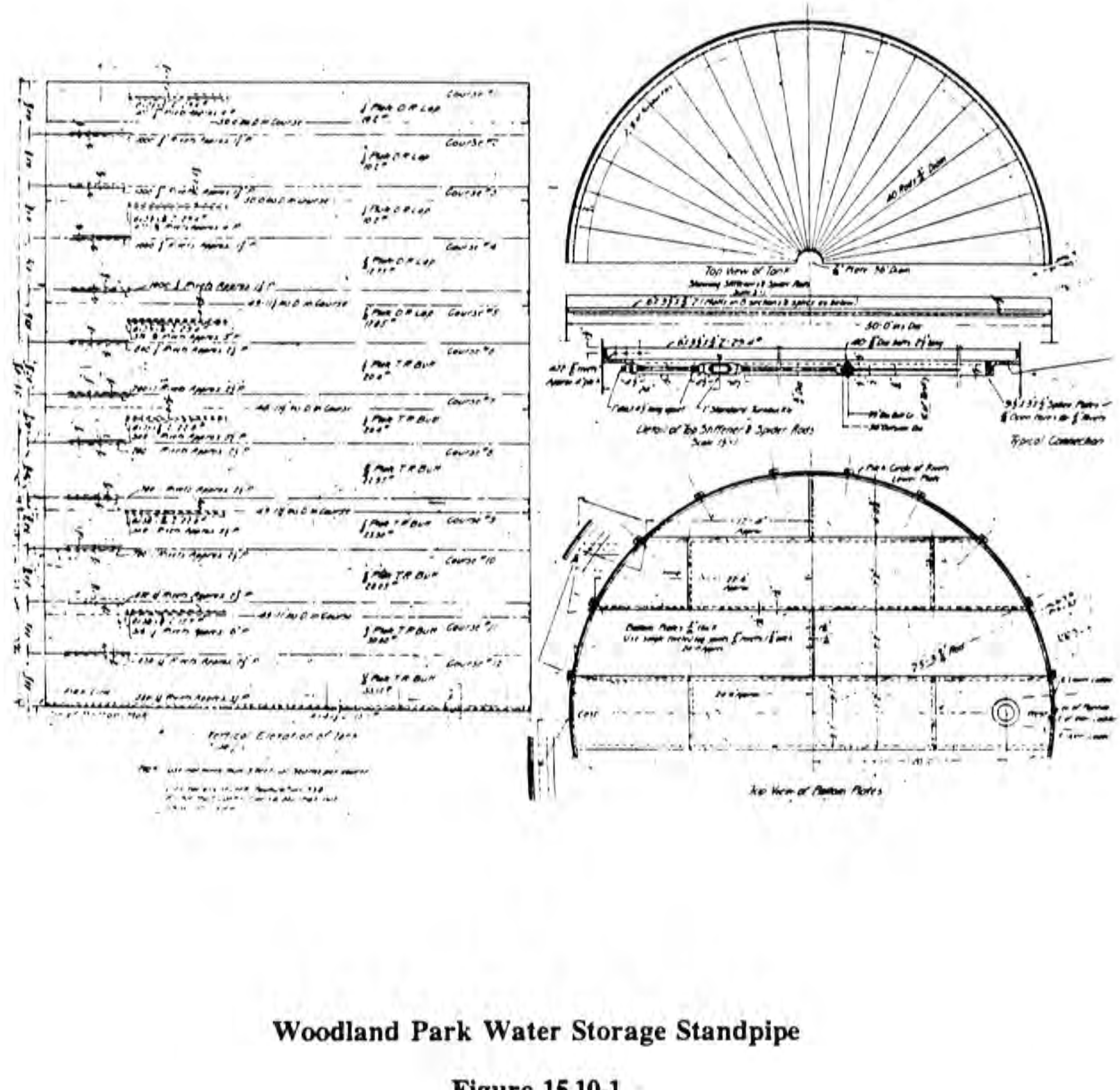
Critical Member or Location	Stress Ratio Against Failure or Yielding	Comments
Shell	<0.3	Adequate
Anchor Bolts	3.39	Failure
Soil	<0.4	Adequate

Standpipe Status: **Not Operable**

Table 15.10-3

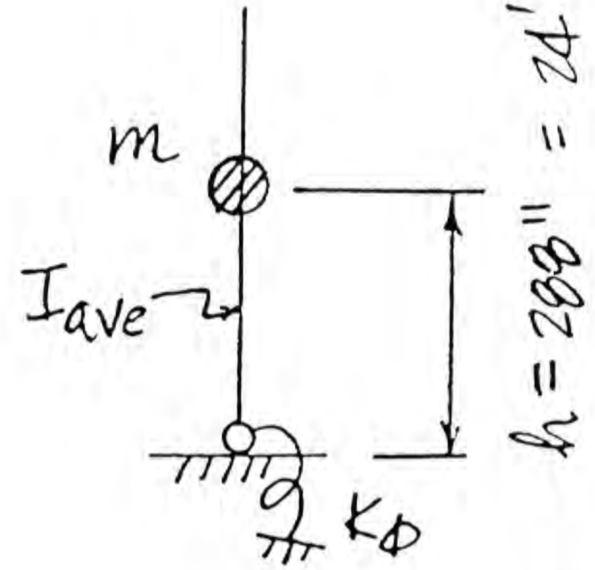
Woodland Park Water Storage Standpipe (High Priority)
Design Base Shear Comparison

1988 UBC Code	2,039 kips
Class I Earthquake	3,997 kips
Class II Earthquake	2,478 kips



Woodland Park Water Storage Standpipe

Figure 15.10-1



$K_\phi = 2.30E9$ "K/rad

$m = 17.1$ K-sec²/in

Woodland Park Water Storage Standpipe
Mathematical Model

Figure 15.10-2

15.11 Summary and Conclusions

15.11.1 Geotechnical

The standpipes are located in areas that are not susceptible to earthquake-induced hazards such as liquefaction or landsliding. Minor surficial landsliding may occur outside of the foundation of the S.W. Trenton standpipe; however, any soil instability in this area is not anticipated to affect the integrity of the standpipe. As there is a potential of fill soils underlying the standpipes located at S.W. Barton, S.W. Charleston, and Volunteer Park, subsurface explorations and geotechnical engineering studies should be accomplished in conjunction with any remedial treatment of these facilities to confirm the adequacy of the foundation soils.

15.11.2 Structural

The Seattle Water Department's water storage standpipes have been evaluated for seismic vulnerability when subjected to Class I and Class II earthquakes.

The evaluation was hampered by missing information. No original design calculations were available for review, the structural drawings were incomplete, and material test data was limited or non-existent. These inconveniences are to be expected when reviewing structures 56 to almost 90 years old, such as these.

In order to proceed with the vulnerability assessment, many assumptions had to be made at every step. While our experience indicates that these assumptions were reasonable, conclusions reached should be interpreted as our professional opinion.

As shown in Table 15.11-1, out of the nine standpipes reviewed all but SW Trenton North and South will undergo severe damage and are expected to be inoperable under the Class I earthquake. The standpipe with the highest vulnerability is Volunteer Park. When the standpipes are subjected to the Class II earthquake, SW Barton and SW Trenton are expected to remain fully operable, while the others are expected to be inoperable.

Table 15.10-2 presents a summary of cost estimates prepared for the seismic upgrade of the standpipes. Included in the engineering costs, in addition to the design engineering costs, are those costs associated with architectural considerations, planning and review board meetings, direct costs, and dealing with anticipated interferences such as private property boundaries, underground utilities, and other anticipated physical constraints encountered in upgrade projects such as these.

Table 15.11-1

Summary Operability Assessment Standpipes

Standpipe	Facility Priority	Class I Earthquake	Class II Earthquake
SW Barton	High	Not Operable	Operable
Charlestown	High	Not Operable	Not Operable
Foy	High	Not Operable	Not Operable
Queen Anne #1	High	Not Operable	Not Operable
Queen Anne #2	High	Not Operable	Not Operable
SW Trenton North	High	Operable	Operable
SW Trenton South	High	Operable	Operable
Volunteer Park	High	Not Operable	Not Operable
Woodland Park	High	Not Operable	Not Operable

Table 15.11-2

Summary Cost Estimate
Standpipes
(Based on 1989 Dollars)

Standpipes	Estimated Costs						Accuracy of Estimate
	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total	
SW Barton	$ 30,000	$ 23,000	$ 60,000	$ 113,000	$ 4,900	$ 117,900	±40%
Charlestown	65,000	50,000	249,000	364,000	20,200	384,200	±40%
Foy	65,000	50,000	249,000	364,000	20,200	384,200	±40%
Queen Anne #1 & #2	107,000	79,000	525,000	711,000	42,500	753,500	±40%
SW Trenton North	0	0	0	0	0	0	
SW Trenton South	0	0	0	0	0	0	
Volunteer Park	200,000	144,000	1,825,000	2,169,000	148,000	2,317,000	±40%
Woodland Park	65,000	50,000	249,000	364,000	20,200	384,200	±40%
Total Cost	$532,000	$396,000	$3,157,000	$4,085,000	$256,000	$4,341,000	

16.0 EMERGENCY RESPONSE PLAN REVIEW

The Seattle Water Department Emergency Response Plan was reviewed for adequacy and completeness pertaining to seismic vulnerability considerations. Our review indicated that the current plan includes most of the basic elements required in a document of this nature. Our recommendations are as follows:

16.1 Earthquake Damage Detection and Isolation

- In order to expedite damage assessment following a disaster, it is recommended that a service agreement be in place for helicopter reconnaissance service.

- It is recommended that the Seattle Water Department consider establishing service agreements with structural experts to facilitate their participation in facility damage surveys following a major earthquake.

- It is recommended that the Seattle Water Department review and expand upon the listing of local suppliers of pumps and motors provided under the Resources section of the Emergency Response Plan on page D-72.

- Water loss, resulting from damage to the water distribution network can be mitigated if sufficient isolation values are installed and operable following a strong earthquake. As discussed in Sections 13.13 and 13.14, it is recommended that personnel from the Water Department review the seismic hazard map and liquefaction hazard map overlays (Figures 4.3-1 and 4.3-2) with respect to the SWD facilities to identify those transmission, distribution and supply lines that pass through areas susceptible to geologic hazards (i.e., landsliding or liquefaction). This information would aid the Water Department in ensuring that isolation valves are properly installed on either side of the susceptible areas and that a sufficient stockpile of replacement material is maintained for those vulnerable segments of pipeline.

  Particular attention should be focused on those 11 pipeline locations summarized as having a moderate to high hazard in Table 13.16-1, as well as on the underwater burial crossings for the West Seattle, Tolt, and S.W. Spokane Street pipelines.

  Dependency solely on telemetry phone lines and network power supply for the operation of these isolation valves should be avoided. Manual operation of selected valves should be considered as part of the emergency operational procedures.

- It is recommended that hydrants on each side of the pressure zone lines be used to provide for portable pump connections to pump from a lower zone to a higher zone under an emergency. Also, it is recommended that SWD consider the use of bypass pump connections on major distribution lines as they pass through other pressure zones.

16.2 Restoration Priorities

The restoration priorities after an earthquake are a function of the magnitude of the disaster and the impact that it has on particular systems. The basic general guidelines to be used in establishing restoration priorities are listed below in decreasing order of importance:

1) Fire Fighting: It is recommended that the Seattle Fire Department review the expected seismic performance of the Seattle Water Department Facilities and correlate that to their projected water demands following an earthquake. There is a strong possibility that the Seattle Fire Department will have to rely on alternative water supplies such as bodies of fresh water (e.g., Lake Washington) and designated emergency water supplies. Typically, water demand peaks immediately following a strong earthquake because of fire fighting needs. On the other hand, water supply capacities drop following a strong earthquake because of damage to the facilities.

2) Structural Safety: Immediately following a strong earthquake, some facilities may be damaged and on the verge of collapse. It may be necessary to vacate those facilities, or operate them at a reduced capacity in order to avoid potential failures. In case a major operational facility needs to be vacated (e.g., the Dexter Horton building), a suitable backup facility location should be arranged for and identified in the Emergency Response Plan.

3) Public Health: Facilities such as hospitals require a continuous supply of water. Although the hospitals are responsible for providing their own emergency backup supply of water in the event of a major disaster, the SWD should consider a means of providing portable water supplies to hospital areas, e.g. through the use of portable water trucks. This is discussed in more detail below under Community Consumption.

4) Community Consumption: Major metropolitan areas affected by strong earthquakes have found that water trucks and portable water tanks provide a valuable resource in maintaining an emergency water supply system. It is recommended that a survey be made of the supplies of available water trucks and portable water tanks in the Puget Sound area, and that service agreements be put in place. Service agreements in place prior to the disaster occurrence will help to mitigate price gouging practices and help capture what will be a scarce supply in a time of high demand. Alternatively, the SWD could consider purchasing a few water trucks which could also be used whenever the shutdown of supply lines for maintenance requires the trucking in of water.

5) Other Water Demands: The prompt restoration of service for industrial and commercial needs is important in order to minimize the economic impact of the disaster on the community.

16.3 Repair Inventories

Typical spares and supplies to be stockpiled included replacement piping, structural steel components, concrete, reinforcing steel, portable pumps, generators, portable disinfection equipment and construction equipment.

Section 13.13 provides guidelines for estimating the required repair inventories for the transmission, distribution and supply pipelines, and Section 13.16 discusses expected damage to underwater burial pipeline crossings. It is estimated that 0.3 to 0.5 breaks per mile could take place in a Class II and a Class I earthquake, respectively. A reasonable inventory of pipe replacement segments, couplings, welding materials and equipment should therefore be stockpiled in order to repair pipeline breaks.

Because it will not be feasible to stockpile everything necessary to cope with a large-scale emergency, it is recommended that certain key and long-lead time items be stockpiled and that sources be identified for supplementation of repair items on a relatively short notice during an emergency.

16.4 Coordination of Recovery Efforts

- The focal point of technical and administrative activities for the Seattle Water Department is the Dexter Horton Building in downtown Seattle. Because of its age and type of construction, this building may not be an appropriate site to serve as a focal point for emergency operations during a major disaster. It may not be feasible to harden the building to the extent necessary and there is also a question of its accessibility and the communications and interactions with the line and support divisions that will be required during an emergency. It is therefore recommended that the SWD consider establishing an alternative to this site as a base for emergency operations.

- It is recommended that training on emergency preparedness and response be provided to SWD employees. This training should include SWD's Emergency Response Plan as well as home safety issues. If SWD employees can avoid or minimize seismic vulnerability in their own homes, their availability for work-related emergencies will increase.

- A key element for the continued operation of a major organization is it's ability to maintain payroll and accounts payable functions. This can be achieved as part of the objectives of an Administrative Services Emergency Preparedness and Response Plan. The development of such a plan is recommended.

- The adequacy of SWD's communications capabilities was reviewed with SWD personnel. There is currently a radio communication system in place for the Seattle Water Department. However, this system is dependent on grid power supply and telephone lines and is therefore vulnerable to earthquakes. Both telephone lines and grid power are anticipated to be out of order following a major earthquake. Because of

this, it is expected that the current communication system will also be out of operation following a major earthquake.

In addition to its seismic vulnerability, the current system also has problems in that it is a long wave-length system (48 MHz) and the signal cannot be repeated. Because of this, it cannot reach low lying areas such as in East King County, because of topography problems.

There is also a microwave system in place. However, this system is used exclusively for a video signal to monitor reservoirs and water quality. This system is limited to its current use by its Federal Communications Commission license.

There are currently a couple of ongoing efforts aimed at improving SWD's radio communications situation. The Department of Administration Services is working on a plan for a city-wide system and is looking into the possibility of replacing the 48-MHz system with an 800-MHz one. A city-wide Disaster Management Committee has been reviewing these communications issues and is reaching similar conclusions.

It is recommended that the ongoing efforts to improve the communication systems be continued and supported. The new system should minimize dependence on grid power supply, avoid the dependence on phone lines, and have built-in redundancy.

16.5 Mutual Aid Agreements

It is recommended that mutual aid agreements be established with other regional water suppliers to provide water, supplies, equipment, spares and crews in the event of an emergency.

16.6 Review of City of Seattle Disaster Readiness and Response

The City of Seattle Disaster Readiness and Response plan was reviewed for compatibility with the Seattle Water Department's Plan. The review was limited to earthquake vulnerability issues. The following is a summary of our review:

- The current City of Seattle Plan (CSP) is dated 1986. It is recommended that this plan be reviewed and updated on an annual basis.
- The CSP designates the Water Department's Operations Control Center as an alternate site for the Emergency Response Center. Cygna's study indicates this building is adequate to withstand seismic hazards.
- Section G of the CSP contains a responsibilities matrix. If fails to explicitly include the Seattle Water Department.

- Also in Section G of the CSP, the Engineering Department responsibilities fail to include emergency food services (drinking water) and medical liaison.

- Section I.B.3.a of the CSP addresses public service limitations to be expected in a natural disaster. It fails to mention the potential loss of water supply and its subsequent fire, health and consumption consequences.

- Section II.L.6 covers the King County Office of Emergency Management responsibilities. Mutual aid agreements for some departments are mentioned, however, there is no mention of mutual aid for the Seattle Water Department.

In summary, the City of Seattle Disaster Readiness and Response Plan does not incorporate water supply disruption considerations in an explicit manner. As the vulnerability study for the Seattle Water Department facilities indicates, there will likely be a loss of water supply to portions of the City's service area as a result of a severe earthquake. This expected water supply disruption is consistent with the experience of other metropolitan areas throughout the world which have experienced major earthquakes. The seismic vulnerability of the Seattle Water Department's facilities is greater than most other cities of its size because of the significant potential for earthquakes in the Puget Sound area, the age and design of its facilities, and its reliance on a number of elevated tanks and standpipes situated throughout the city for water storage.

17.0 RECOMMENDATIONS FOR EXISTING FACILITY AND PIPELINE UPGRADES

Provided in this section is a prioritized list of the sequence by which both the geotechnical and structural upgrade recommendations should be implemented for the SWD facilities and pipelines evaluated as part of this seismic vulnerability study.

This section is divided into two parts: a) Major Projects that will need to be performed by outside contractors; and b) Minor Maintenance Projects that can be performed by SWD personnel. Each of these sections contains its own set of prioritized implementation guidelines. The actual sequence and schedule by which these upgrade projects will take place will, of course, depend on the SWD's internal budgetary and manpower constraints. It is expected, however, that these upgrades will need to be implemented over a 5 to 7 year period of time.

17.1 Major Projects

In this section, a prioritized ranking is provided for all of the major facility upgrade projects which were identified as being necessary as a result of this study. Those items which are classified as Major Projects typically cost more than $10,000 and therefore require a significant amount of capital expenditure coupled with a schedule which allows sufficient time for planning and design activities.

A prioritized ranking of those facility upgrades which are classified as Major Projects is provided in Table 17-1, along with an estimated cost for each of the upgrades. The estimated cost is inclusive of additional investigations, engineering, construction engineering and construction.

In ranking these projects, an attempt has been made to establish a prioritization scheme that is based upon those facilities which are determined to be most highly vulnerable and would have the most impact on either life safety or system functionality following a major earthquake.

An estimated schedule for both the design and construction phases for selected major projects (or categories of projects) is provided in Table 17-2.

17.2 Minor Maintenance Projects

Those facility upgrades which are identified as Minor Maintenance Projects can generally be accomplished by the SWD's maintenance and operations personnel and will provide a fairly significant increase in the safety of a given facility for a nominal investment of labor and time.

A listing those items classified as Minor Maintenance Projects is provided in Table 17-3, along with an estimated cost for each of the upgrades. The estimated cost is inclusive of additional investigations, engineering, construction engineering and construction.

It is recommended that the Minor Maintenance Projects be undertaken as soon as reasonably practicable. In implementing these projects, the following general priorities should be followed: High Priority - anchor WOCC control room cabinets and planters; secure all chlorine tanks at chlorination facilities; Medium Priority - anchor all mechanical/electrical equipment and fuel tanks; Low Priority - anchor all lockers, cabinets and storage racks.

17.3 Summary

In Table 17-4 is provided a summary of the estimated costs to upgrade the SWD facilities which were reviewed as part of this seismic vulnerability study. The costs are broken down into discrete estimates for engineering, construction engineering, and construction. Estimated costs for performing additional recommended investigations are also included in this summary table. The total cost estimate for both the additional investigations and seismic upgrades is $10,314,000.

Table 17-1

Major Projects

Water Operations Control Center		Cost
1.	Warehouse (Section 10.4) - additional investigations; strengthen connections; add shear wall	$343,400
2.	Pipe/Carpentry Shop (Section 10.6) - strengthen connections and pile ties	$112,300
3.	Vehicle Maintenance/Storage (Section 10.7) - replace south wall	$190,800
Chlorination Facilities and Pump Stations		
4.	Beacon Hill Water Quality Bldg. (Section 11.4) - additional investigations; tie roof-walls and construct shear panels	$97,800
Control Works		
5.	Control Works Bldg. (Section 9.0) - additional investigations; infill windows with reinforced concrete	$184,300
Major Supply and Distribution Pipelines		
6.	West Seattle (Section 13.7) - install tie-down straps	$47,600
7.	Mercer Island (Section 13.11) - install tie-down straps and re-support pipeline (re-support cost not included)	$99,900
8.	Cedar River (Section 13.15) - install tie-down straps	$32,800
9.	Eleven Locations with Moderate Hazard Potential (Table 13.15-1) - additional investigations and geotechnical upgrades (as required)	$198,900
Lake Youngs		
10.	Office Bldg. (Section 8.3) - additional investigations	$3,000
11.	Chlorination Bldg. (Section 8.7) - additional investigations	$4,000
12.	Corrosion Treatment Bldg. (Section 8.8) - additional investigations	$3,100

Table 17-1

Major Projects
(Continued)

Elevated Tanks and Standpipes		**Cost**
13.	Volunteer Park (Section 15.9) - strengthen brick facade and tank foundation	$2,317,000
14.	Queen Anne #1 and #2 (Sections 15.6 and 15.7) - add braces and footing	$753,500
15.	Magnolia Bluff (Section 14.4) - increase area of braces and columns	$500,200
16.	S.W. Myrtle #2 (Section 14.7) - increase area of braces and columns	$585,400
17.	Richmond Highlands #2 (Section 14.9) - strengthen columns and braces	$626,300
18.	Richmond Highlands #1 (Section 14.8) - strengthen columns and braces	$491,200
19.	Charlestown (Section 15.4) - add braces and footing	$384,200
20.	Foy (Section 15.5) - add braces and footing	$384,200
21.	Beverly Park (Section 14.3) - brace interior columns	$305,600
22.	Maple Leaf (Section 14.5) - strengthen bracing, anchorage and connections	$428,500
23.	S.W. Barton (Section 15.3) - add anchor bolts	$117,900
24.	S.W. Myrtle #1 (Section 14.6) - strengthen columns, braces and connections	$433,300
25.	Woodland Park (Section 15.10) - add braces and footing	$384,200
26.	Landsburg Elevated Tank (Section 6.6) - increase size of footings (repair work may be done as part of fish ladder project)	$48,720

Table 17-1

Major Projects
(Continued)

Tolt South Fork		**Cost**
27.	Turbine Bldg. (Section 5.7) - investigate roof-wall connection	$1,600
28.	Screenhouse (Section 5.8) - tie roof-walls; infill windows	$60,970
29.	Chlorination Bldg. (Section 5.9) - tie roof-walls; infill windows	$67,720
30.	Corrosion Treatment Bldg. (Section 5.10) - investigate roof penetration	$3,500
Landsburg Diversion		
31.	Screenhouse (Section 6.3) - additional investigations; infill windows	$82,700
32.	Chlorination Bldg. (Section 6.4) - tie roof-walls	$82,510
33.	Tunnel Gate House (Section 6.8) - additional investigations	$3,600
Chlorination Facilities and Pump Stations		
34.	Fairwood (Section 11.12) - additional investigations; tie roof-walls	$40,960
35.	Burien (Section 11.9) - tie roof-walls; strengthen roof	$60,960
36.	Bitter Lake (Section 11.5) - tie roof-walls; reinforce walls & roof	$98,900
37.	S.W. Spokane St. (Section 11.31) - additional investigations; tie roof-walls; strengthen roof	$74,560
38.	Interbay (Section 11.18) - tie roof-walls; reinforce walls & roof	$74,000
39.	Broadway (Section 11.8) - additional investigations; tie roof-walls; strengthen roof	$78,260
40.	Roosevelt (Section 11.30) - tie roof-walls; reinforce walls & roof	$85,160
41.	Maplewood (Section 11.25) - additional investigations; tie roof-walls	$35,360
42.	Green Lake (Section 11.16) - additional investigations	$4,000

Table 17-1

Major Projects
(Continued)

Chlorination Facilities and Pump Stations (Cont'd)

43.	Lake Forest (Section 11.19) - additional investigations	\$3,000
44.	Lincoln Park (Section 11.21) - additional investigations	\$5,000
45.	Maple Leaf P.S. (Section 11.24) - additional investigations	\$3,600
46.	Volunteer Park Gate House (Section 11.37) - additional investigations	\$5,000
Miscellaneous Facilities		
47.	Beacon Reservoir Gate House (Section 12.1) - additional investigations	\$3,000
Water Operations Control Center		
48.	Flammable Storage Building (Section 10.5) - replace building	\$97,900
Major Supply and Distribution Pipelines		
49.	Six Locations with Low Hazard Potential (Table 13.15-1) - additional investigations and geotechnical upgrades (as required)	\$98,300
	Total Cost	\$ 10,148,680

Table 17-2

Design and Construction Schedule for Selected Major Projects

	Project	Design Phase (Weeks)	Permit Phase (Weeks)	Construction Phase (Weeks)
1.	Water Operations Control Ctr.	24-30	12-16	24-30
2.	Treatment/Pump Station Bldg.(1)	6-8	6-10	8-12
3.	Elevated Tank(1)	8-12	12-16	12-16
4.	Standpipe(1)	8-10	12-16	12-16
5.	Volunteer Park	24-30	12-16	32-52
6.	Queen Anne #1 & #2	16-20	12-16	24-30

(1) Design/construction times are provided for a "typical" structure

Table 17-3

Minor Maintenance Projects

	Cost
Tolt South Fork	
1. Duvall Shop (Section 5.3) - anchor lockers	$750
2. Screen House (Section 5.8) - secure wood cabinets; bolt generator to slab; secure lean-to roof	$3,180
3. Chlorination Bldg. (Section 5.9) - secure chlorine tanks and lockers	$2,640
4. Corrosion Treatment Bldg. (Section 5.10) - anchor lockers and panel	$1,380
5. Tolt Standpipe (Section 5.11) - verify analysis assumptions	$300
6. Maintenance Bldg. (Section 5.12) - anchor rack and oxygen tanks	$960
Landsburg Diversion	
1. Chlorination Bldg. (Section 6.4) - secure chlorine tanks	$2,640
2. Generator Bldg. (Section 6.5) - anchor cabinet and generator fuel tank	$2,330
Cedar Falls	
1. Office Shop (Section 7.3) - anchor lockers and storage racks	$3,200
Lake Youngs	
1. Office Bldg. (Section 8.3) - anchor lockers and storage racks; strengthen garage door openings	$14,650
2. Equipment Maintenance Bldg. (Section 8.4) - anchor wood cabinet	$110
3. Pipe Storage Area (Section 8.6) - strap/secure replacement pipe	$5,250
4. Chlorination Bldg. (Section 8.7) - secure chlorine tanks	$3,650
5. Corrosion Treatment Bldg. (Section 8.8) - anchor panel	$1,640

Table 17-3

Minor Maintenance Projects
(Continued)

Water Operations Control Center	**Cost**
1. Administration Bldg. (Section 10.3) - anchor control room cabinets; secure planters to bldg.; anchor boiler, shelves, cabinet, tank	$19,500
2. Warehouse (Section 10.4) - brace storage racks; store heavy items on lower shelves; secure chlorine cylinders	$23,200
3. Pipe/Carpentry Shop (Section 10.6) - Verify/install equipment anchors	$2,100
4. Vehicle Maintenance/Storage (Section 10.7) - anchor storage racks; store heavy items on lower shelves	$21,600
Chlorination Facilities and Pump Stations	
1. S. Augusta St. (Section 11.3) - anchor motor control center	$640
2. Beacon Hill Water Quality Bldg. (Section 11.4) - secure chlorine tanks; strap chlorine analyzers; anchor lockers & cabinets	$5,700
3. Bitter Lake (Section 11.5) - secure chlorine tanks; verify/anchor motor control center	$3,100
4. Bothell Way (Section 11.6) - anchor A/C unit	$960
5. Boulevard Well (Section 11.7) - anchor transformer	$640
6. Broadway (Section 11.8) - secure chlorine tanks	$2,640
7. Burien (Section 11.9) - verify/anchor motor control center	$640
8. Fairwood (Section 11.12) - verify/anchor control panel	$640
9. First Hill (Section 11.13) - verify/anchor control panel	$640
10. Foy (Section 11.14) - anchor eastern control panel	$640
11. Green Lake (Section 11.16) - secure chlorine tanks	$2,640
12. Highland Park (Section 11.17 - verify/anchor motor control equipment	$640
13. Lake Forest (Section 11.18) - secure chlorine tanks	$2,640

Table 17-3

Minor Maintenance Projects
(Continued)

Chlorination Facilities and Pump Stations (Cont'd)	Cost
14. Lake Hills (Section 11.20) - verify/anchor control panel	\$640
15. Maple Leaf (Section 11.23) - secure chlorine tanks	\$2,640
16. Maplewood (Section 11.25) - verify/anchor control panel	\$640
17. North City (Section 11.27) - verify/anchor motor control center; anchor A/C unit	\$1,280
18. Riverton Heights (Section 11.29) - anchor transformer	\$640
19. Roosevelt (Section 11.30) - verify/anchor control panel	\$640
20. S.W. Spokane St. (Section 11.31) - verify/anchor motor control center	\$640
21. S.W. Trenton (Section 11.34) - secure chlorine tanks	\$2,640
22. View Ridge (Section 11.35) - verify/anchor A/C unit	\$640
23. Volunteer (Section 11.36) - verify/anchor control panel	\$640
24. Warren Ave. (Section 11.38) - verify/anchor control panel	\$640
25. West Seattle (Section 11.39) - secure chlorine tanks; install concrete valve pedestals; anchor motor control centers	\$10,400
Miscellaneous Facilities	
1. Beacon & Jefferson Field House (Section 12.5.2) - stop corrosion of rebar and valve	\$2,450
2. Cedar R. Wye Vault (Section 12.5.4) - stop corrosion of valves	\$2,450
3. Mercer Island Vault (Section 12.5.5) - re-mortar walls	\$4,100

Table 17-3

Minor Maintenance Projects
(Continued)

Pipelines	**Cost**
1. Tolt SF Supply Line (Section 13.8.3) - verify construction records	$500
2. ESSL (Section 13.10.4) - remove tree	$1,000
3. ESSL (Section 13.10.6) - remove tree	$1,000
4. Miscellaneous Supply Lines (Section 13.14.11) - review fill placed over pipeline alignment	$500
Total Cost	$ 165,320

Table 17-4

Summary of Upgrade Recommendations
Seismic Reliability Study
(Based on 1989 Dollars)

Facility	Estimated Costs						
	Additl. Invest.	Engineering	Construction Engineering	Construction	Subtotal	Sales Tax (8.1%)	Total
Tolt	$ 17,100	$ 38,000	$ 25,900	$ 57,300	$ 138,300	$ 4,700	$ 143,000
Landsburg	11,600	54,700	38,800	108,600	213,700	8,800	222,500
Cedar Falls	-0-	300	200	2,500	3,000	200	3,200
Lake Youngs	10,100	8,300	2,600	13,300	34,300	1,100	35,400
Control Works	30,000	32,800	24,200	90,000	177,000	7,300	184,300
Water Operations	2,000	133,000	101,200	531,500	767,700	43,100	810,800
Treatment/Pump Stations	47,600	185,900	94,200	354,400	682,100	28,700	710,800
Misc. Facilities	3,000	1,000	400	7,000	11,400	600	12,000
Pipelines	97,000	92,000	85,000	191,000	465,000	15,500	480,500
Elevated Tanks	-0-	499,000	383,000	2,302,000	3,184,000	186,500	3,370,500
Standpipes	-0-	532,000	396,000	3,157,000	4,085,000	256,000	4,341,000
Totals:	$218,400	$1,577,000	$1,151,500	$6,814,600	$9,761,500	$552,500	$10,314,000

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19.0 GLOSSARY

Term	Definition
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSYS	engineering analysis program
API	American Petroleum Institute
ATC	Applied Technology Council
BFV	butterfly valve
Class I eq	One-in-100 year earthquake
Class II eq	One-in-500 year earthquake
c.m.u.	concrete masonry units
CRSL	Cedar River Supply Line
CSP	City of Seattle Plan
Cygna	Cygna Consulting Engineers
EMR	Emergency Response Plan
ESSL	Eastside Supply Line
g	acceleration of gravity
GV	gate valve
kips	1,000 pounds
ksf	kips per square foot
ksi	kips per square inch
LYA	Lake Youngs Aqueduct
LYSL	Lake Youngs Supply Line
MBE	Minority Business Enterprise
MCE	Maximum Credible Event
MIPL	Mercer Island Pipeline
MLPL	Maple Leaf Pipeline
psf	pounds per square foot
SWD	Seattle Water Department
TPL	Tolt Pipeline
TS	tube steel
UBC	Uniform Building Code
USGS	U. S. Geological Survey
WBE	Women Business Enterprise
WOCC	Water Operations Control Center
WSPL	West Seattle Pipeline